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Thesis  
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# 49

THE UNIVERSITY OF ALBERTA

STABILIZATION OF A HIGHLY PLASTIC CLAY

WITH LIME AND POZZOLAN

A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES

IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE

OF MASTER OF SCIENCE

DEPARTMENT OF CIVIL ENGINEERING

BY

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## ABSTRACT

Lime and pozzolan added in the proper proportions to clay soils form mixtures that compacted to a dense state are stabilized to resist deformation under applied stresses. Ion exchange in the calcium-rich environment causes aggregation of the particles into friable mixes which after compaction develop strength through carbonation and pozzolanic cementation. The frost susceptible pozzolanic reaction can sometimes be accelerated by introducing traces of bases or basic salts into the clay-lime-pozzolan mixtures.

A test program measured the 28-day strength and the durability of mixtures with additive contents ranging from 5 to 20 per cent and with lime-pozzolan ratios distributed from 10:0 to 1:9. Each mixture of highly plastic clay, calcitic hydrated lime, and pozzolanic Diamond City Shale, was compacted to two different densities by varying the compactive effort.

Lime-Pozzolan mixtures containing 20 per cent additive at lime-pozzolan ratios greater than unity, when compacted to 95 per cent of the modified AASHTO density increased the immersed strengths of the clay from zero to 250 psi. Such variables as the additive content, lime content, lime-pozzolan ratio, moulding moisture content, rotting time, compactive effort, curing time and temperature, and soaking time affected the strength of the stabilized mixes.



A laboratory durability test is required which will duplicate the worst possible failure conditions experienced in field projects. The use of lime to stabilize clays in Alberta looks most promising when used in conjunction with other agents such as trace chemicals, activated bitumen or portland cement.





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## CHAPTER I

### INTRODUCTION

Soil Stabilization implies the improvement of a soil so that it may offer increased resistance to deformation. Mechanical stabilization has been the usual method of improving the strength of the bases and subbases of highways. This method entails the careful selection of the quality and grading of natural granular materials, and compaction to form them into the densest state possible. In the great plains region of central North America, the supply of suitable aggregate is scarce and irregularly distributed. The large expenditures entailed in hauling, crushing and screening gravels have increased the cost of mechanical stabilization to the point where other forms of stabilization may realize savings in the capital and maintenance costs of highways.

Alternative methods of stabilization are (1) by the addition of additives to the natural soils, (2) by the process of electroosmosis, or (3) by the application of high temperatures. The first method has been the one chosen as the most practical and economical substitute for mechanical stabilization. By means of additives, bases can be built of poorer quality aggregates, the strength coming from a bonding agent rather than from interparticle friction. The aggregate in the subbase can often be replaced by improving the subgrade material



with the addition of a bonding agent.

Soil-cement, the most common of the stabilization processes, is a mixture of cement and water with a pulverized soil. The mixture is compacted to a high density and cured at constant water content. It possesses high strength and durability. The cement is difficult to mix into a fine-grained soil which has not been pulverized to pass a Number 4 sieve.<sup>1</sup>

Soil-bitumen mixtures provide good stabilized bases in areas where gravels are scarce but sands and silts are abundant. The bitumen provides cohesive strength to the sand without appreciably reducing the frictional resistance. Bitumens also act as waterproofing agents for cohesive soils. They maintain a lower moisture content and subsequently higher strength in the soil. Bitumens cannot be used on highly plastic and medium plastic clays because of the resistance of the clays to pulverization.

Lime and lime-pozzolan stabilization has been particularly effective in regions with hot climates, because the strength is increased by the heat and detrimental frost action is not a threat. Lime assists in pulverizing the highly cohesive clays, increasing their workability; and the drying action of the lime allows more working days for the construction forces.

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<sup>1</sup>U. S. Standard Sieve size.





Other chemicals used to stabilize soils include calcium and sodium chloride, lignin derivatives, synthetic resins, ammonium derivatives, phosphates and sodium silicate.

### I.1 STATEMENT OF THE PROBLEM

This thesis was part of a project studying the chemical stabilization of soil; one of the continuing investigations of the Highway Research Program.

The particular study reported in this thesis concerned lime and lime-pozzolan stabilization of a highly plastic clay soil. The principal problems to be considered were:

1. Would the 28-day strengths achieved by stabilized mixtures of local materials be sufficient to warrant their application in a highway pavement structure?
2. Were the stabilized mixtures sufficiently durable to withstand the rigorous climate of Alberta?

### I.2 WORK DONE TO PROVE LIME STABILIZATION

The study consisted of a search of available literature and the performance of a laboratory test program. A brief summary of the published history of lime stabilization is followed by a detailed description of the scope of this thesis.



## History of Lime and Lime-Pozzolan Stabilization

Modern use of lime in roads began after the First World War. Clare and Cruchley (1957) presented a brief history of studies and the conclusions of the early investigators.<sup>1</sup> The great advance in the knowledge of lime stabilization began in Texas. The first large job in that state was composed of the runways and taxiways of an airfield in which the gravel was stabilized with lime. By 1948 there were approximately seventy five miles of lime-stabilized base course existing in Texas. Although Texas now has the most lime stabilized roads of any state, Iowa State University has been the fountainhead of most of the published literature on the subject. The research was conducted at the Iowa Engineering Experiment Station under the sponsorship of the Iowa Highway Research Board, and was supported mainly by funds provided by the Iowa State Highway Commission and the United States Bureau of Public Roads.

The Department of Scientific and Industrial Research, Road Research Laboratory in the United Kingdom has also been carrying out stabilization studies, but the articles by Clare and Cruchley (1957) and Sherwood (1961), and the unpublished research note by Dumbleton and Ross (1960) were the only

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<sup>1</sup>References are cited by indicating the author and the year of publication. The references are contained in the bibliography at the conclusion of this thesis.



publications available for this report.

A valuable annotated bibliography was included in a report by Herrin and Mitchell (1960) on lime stabilization. A parallel bibliography on lime-pozzolan-aggregate mixtures was prepared by Hollan and Marks (1960).

Field construction practices are outlined in the booklet prepared by the National Lime Association (1954).

### Lime Stabilization Studies in Alberta

Lime stabilization studies at the University of Alberta were initiated by Clark (1958), who confirmed that it could be used to reduce the plasticity of the fines in a submarginal gravel. Millions (1959) continued the investigation of the effects of additives on the characteristics of gravels. Harvey (1960) studied the effect of freezing and thawing upon the plasticity of gravel fines treated with lime and flyash. Each investigator worked with small quantities of lime, the maximum percentage of lime used was  $3\frac{1}{2}$  per cent of the dry weight of the aggregate. The flyash content brought the weight of the lime-flyash additive up to  $10\frac{1}{2}$  per cent. These quantities of lime and flyash were identical to mixtures which have given satisfactory results in various States.

Lindberg (1960) ran some moisture content-density curves of lime-pozzolan mortar mixes. His work was a valuable contribution in the improvement of the technique of the present





investigation. A valuable report was also prepared by Forbes (1960), who conducted a series of strength tests and freeze-thaw tests on six different soils of Alberta. Specimens were made in Standard Proctor molds, cured at less than one hundred per cent humidity, and were then broken in an unsaturated condition, to discover the unconfined compressive strength. The difference in the compaction, curing and testing techniques between the work of Forbes and the present investigation enabled each to make a unique contribution to the knowledge of lime-soil stabilization. Forbes (1960) also conducted his freeze-thaw tests in accordance with ASTM D 560-57, except that the specimens were not brushed after every cycle.<sup>1</sup> The present investigation used a different method of evaluating the durability of the mixtures.

The stabilization of a silt with lime and pozzolan is being investigated by Hvozdzanski as another part of the project studying the chemical stabilization of soils, part of a continuing investigation of the Highway Research Program. The present investigation is a companion study into the effects of the same substances on a clay soil. Most of the equipment and technique used in this thesis was developed by Hvozdzanski.

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<sup>1</sup>Details of test found in ASTM (1958). See Bibliography.





## Scope of This Investigation

Verification of conclusions drawn by others. The primary purpose of this investigation was to discover if lime-stabilization was feasible for an Alberta clay. Results from other centers had indicated high strength and durability characteristics were imparted to a soil by lime and pozzolan mixtures if the proper procedures of mixing, compacting and curing were followed. A highly plastic clay was chosen for the tests. The effect on the strength and durability of varying the quantity of additive, the proportions of lime and pozzolan and the density was studied. The strengths which were obtained represented the minimum strength possible from each specimen. A reduction in the time between mixing and compacting, or testing the specimens in an unsaturated state, would probably have resulted in higher strengths.

The reaction of carbon dioxide with calcium hydroxide results in the formation of calcium carbonate, which is a weak cement compared to the calcium silicates formed from the reaction of the lime and pozzolan. The formation of calcium carbonate is also subject to a different set of variables than those which affect the pozzolanic strength. Therefore, the formation of calcium carbonate was prevented to permit a clearer study of the variables which affect the pozzolanic strength.



The recrystallization of calcium hydroxide, another weak cement, was stopped for the same reason.

New fields of study. One would be presumptuous in assuming that any part of this investigation was original. Yet this was the first study of the undrained, saturated strength of a lime-stabilized clay from northern Alberta.

It is believed that both the method and the apparatus used for conducting the freeze-thaw tests were unique.

Lignin derivatives have been tried as stabilizing agents, but a review of the published literature, did not mention their use as a reinforcement for the strength and durability of lime mixtures. Even if negative results were achieved, the combination of an organic substance with the lime yielded valuable information.

This investigation followed the example of Hvozdzanski in using six specimens to establish the strength of each compacted mixture. By using six specimens, confidence in the accuracy of these results was justified when compared with other studies based upon duplicate or triplicate samples.

Outline of thesis. The second chapter presents the results of a search of the pertinent literature which discussed the strength and durability of clays stabilized with lime and pozzolan. The constituents and the probable compounds formed from their interaction are described. A report is given on the variables most likely to affect the strength of clay-lime-pozzolan mixtures. Comments follow on the durability



of stabilized clay-lime and clay-lime-pozzolan mixtures in service. The types of laboratory durability tests are briefly described.

Chapter three is a search of the literature which concerned the accelerators used in this thesis only. The development of strength in lime and pozzolan mixtures is a slow process. If the construction of a stabilized pavement is delayed until late autumn, the treated soil probably does not gain sufficient strength to survive the winter. Chemicals, used in trace quantities, have been used to accelerate the development of strength.

Chapter four describes the test program used in this investigation, the results of which are reported in chapter five. Chapter six is reserved for conclusions and recommendations arising from this study.



## CHAPTER II

### CURRENT KNOWLEDGE OF LIME STABILIZATION OF CLAYS

#### II.1 THE CONSTITUENTS

##### Lime

When calcium carbonate is heated to temperatures in excess of 1100C the compound breaks down into a gas, carbon dioxide, and into calcium oxide or calcitic quicklime. The latter is the unstable form of the compound and when exposed to carbon dioxide, the quicklime slowly reverts to calcium carbonate.

The quicklime hydrates or "slakes" when exposed to water. The product, calcium hydroxide, is sold commercially as hydrated lime.

A dolomitic lime is one which contains a considerable quantity of magnesium oxide. Dolomitic quicklime contains calcium oxide and magnesium oxide. When this lime is slaked the calcium oxide is quickly converted into calcium hydroxide, but the magnesium oxide hydrates much more slowly and the lime is sold with most of the magnesium oxide in the unslaked form. Lime sold in this form is called normal hydrated or monohydrated dolomitic lime. This type of lime, designated as type N, has so far proven to be the most desirable type of lime for the stabilization of soils. The lime industry manufactures a





second type of dolomitic lime, called highly hydrated or dihydrated dolomitic lime. This lime designated as type S, is favored by the plastering industry because all of the magnesium oxide is hydrated before it is marketed. However the dihydrated lime is not as effective as the monohydrated lime in the stabilization of soils. To sum up, a tabulation is presented of the types of lime and their chemical formulas.

Calcitic quicklime	$\text{CaO}$
Calcitic hydrated lime	$\text{Ca(OH)}_2$
Dolomitic quicklime	$\text{CaO} + \text{MgO}$
Dolomitic monohydrated lime	$\text{Ca(OH)}_2 + \text{MgO}$
Dolomitic dihydrated lime	$\text{Ca(OH)}_2 + \text{Mg(OH)}_2$
Limestone or calcium carbonate	$\text{CaCO}_3$

### Pozzolan

Lea's (1956) definition and description of a pozzolan is as follows:

Pozzolans are materials which, though not cementitious in themselves, contain constituents which will combine with lime at ordinary temperatures in the presence of water to form stable insoluble compounds possessing cementing properties. . . natural pozzolanas are for the most part volcanic in origin but include also certain diatomaceous earths. The artificial pozzolanas are mainly products obtained by the heat treatment of natural materials such as clays and shales and certain siliceous rocks and pulverized ash [flyash].

Lea (1956) noted that burnt clay pozzolans were produced at temperatures varying from 600C to 900C "depending on the nature of the clay and the conditions of burning." At 500C



kaolinite loses its combined water, while montmorillonite and illite lose their combined water at lower temperatures. When the water is lost, there is some doubt as to the changes that occur in the clay structure but Lea (1956) thought that a mixture of amorphous silica and alumina was formed.

Although the type of pozzolan used in this investigation was a naturally burned shale, another type, called flyash, deserves mention because of its frequent occurrence in the literature. Flyash is a waste product from the burning of finely divided coal and tons of the material are collected by precipitators installed in large industrial chimneys to prevent air pollution. It is composed of spherical glass particles of silica and alumina and rounded particles of magnetic iron oxide. Flyash approximates a burned clay high in alumina and iron oxide, but the major constituent is glass with quartz as one of the major components.

Both the burned shale type of pozzolan and flyash often contain free carbon as a diluent.

### Clays and Clay Minerals

Stabilization of clays results from changes in the physico-chemical properties of the soil. The individual particles of clay are made up of one or more members of a small group of minerals called clay minerals, plus silica and other relatively inert materials. The clay minerals may be



divided into three main groups: (1) kaolinite and halloysite, (2) montmorillonite and illite, and (3) chlorite.

Kaolinite consists of sheets of a crystalline character held together by hydrogen bonds and does not allow replacement of the silicon and aluminum ions within the crystal lattice. The cation exchange capacity of kaolinite is low.

Montmorillonite often consists of the same components as kaolinite except the quantity and arrangement of the ions are different. The sheets are not bound tightly and water molecules may enter between them causing considerable swelling. The cations within the crystal lattice are neither fixed in number or composition. Therefore, there is often a great variation in the net electrical charge on the particle. Montmorillonite has a large cation exchange capacity as the adsorbed cations are arranged between the sheets as well as along the edges of the lattice.

Illite has a structure similar to montmorillonite except that the sheets are tightly bound by potassium ions and there is considerable replacement of silicon by aluminum within the lattice. These two factors tend to balance the charges within the lattice and reduce the cation exchange capacity to some intermediate value between that of kaolinite and of montmorillonite.

Chlorite has a more complex structure, but since the clay used in this experiment was devoid of chlorite, a description is not necessary.





## II.2 IMMEDIATE REACTIONS BETWEEN THE CONSTITUENTS

### Ion Exchange Phenomena

When lime is added to a clay soil, an immediate reaction takes place between the calcium ions,  $\text{Ca}^{++}$ , and the adsorbed ions on the clay minerals. A brief description of the phenomena of ion exchange will explain the mechanics of the reactions.

The charge deficiency in the structure of the crystal lattice of the clay mineral is invariably negative. Thus positively charged ions, cations, are attracted to the surface of the structural units and are held there in various types of bonding. Grim (1953) described two methods by which the cations are held by the silica-alumina clay mineral units. In kaolinite, the cations are held around the edges of the lattice where the broken bond causes a charge deficiency. In montmorillonite, the ions may penetrate the lattice itself causing substitutions of lower valent ions for those of higher valency.

The cations which are held on and in the lattice structure can be replaced by other cations. For example, the replacement of  $\text{Ca}^{++}$  and  $\text{Mg}^{++}$  by  $\text{Na}^+$  can be achieved by increasing the concentration of  $\text{Na}^+$  in solution. This reaction is reversible, in fact Grim (1953) asserted that "other thing being equal, the higher the valence of the ion, the greater its replacing power and the more difficult it is to replace." That is, as the population of  $\text{Ca}^{++}$  on the surface is diminished, the calcium





ions become more and more difficult to release. On the other hand, sodium ions become easier to release as the degree of saturation with  $\text{Na}^+$  is decreased.

Grim (1953) also mentioned that the replacement of the cations was dependent upon the nature of the anion in solution. For instance, calcium hydroxide gave a different exchange capacity between  $\text{Ca}^{++}$  and  $\text{Na}^+$  than did calcium sulfate.

Hydrogen behaves like an ion of higher valence. Potassium though univalent, is almost irreplaceable, because it happens to have the proper dimensions to fit neatly into the cavities of the lattice.

For ions of equal valence, the ion with the largest hydrated volume is the easiest to replace. Heating beyond 100C also affects the exchange capacity of the clay minerals.

The exchange capacity of a clay may be reduced by the presence of large, flat organic ions. These ions spread over large areas of the surface and over more than one exchange position.

The mechanics which occur when lime is first mixed with the soil was described by Herrin and Mitchell (1960) as a combination of two conditions. Firstly, a base exchange takes place in which the calcium ions replace most of the weaker metallic ions, such as sodium and hydrogen, as the adsorbed cations. Secondly, the high concentration of calcium ions in the solution causes a high degree of saturation of calcium ions



on predominantly calcium clays. As outlined by Lambe (1960) the higher valence, smaller size, and the higher degree of saturation of the calcium ion combine to reduce the repulsive electrical forces between the particles. The particles are drawn together (flocculate) and the intrinsic strength of the soil mass is increased.

### The Effect of Lime on the Plasticity of a Clay

The addition of lime to a plastic soil causes the smaller particles to flocculate, forming more friable aggregates. In this condition, the soil is more easily worked and dries quickly.

The change in the plastic limit caused by the addition of lime to a soil, has been shown by Harvey (1960) and others to be a reliable indicator of the reduction in the plasticity of the soil. Hilt and Davidson (1960) concluded that a unique relationship existed between the plastic limit and the unconfined compressive strength of some clay soils. Each clay required a certain amount of lime to reduce the plasticity and until this requirement, called the lime fixation capacity, was satisfied the soil showed little gain in strength. The lime fixation capacity could serve as an excellent minimum or maximum depending upon whether the intent was to design for strength or for improved workability of lime mixtures.

## II.3 THE MECHANICS OF STRENGTH DEVELOPMENT

The contemporary theory of strength development, as



recorded by Davidson and Handy (1960), comprises three concurrent processes. They are ionic bonding, pozzolanic cementation and carbonation. According to Clare and Cruchley (1957) some of the strength may also come from recrystallization of the calcium hydroxide with an increase in temperature. However, this possible source of strength has been ignored in current literature from the United States and Canada.

The early strength is derived from ionic bonding and carbonation. The pozzolanic reaction is believed to be a slow process and it imparts a long-term increase in strength to lime-soil mixtures. It is subject to such variables as temperatures, nature of pozzolan, surface area and time. Little is known about the manner in which the lime combines with the alumina and silica in the clay minerals.

Eades and Grim (1960) did x-ray diffraction and differential thermal analyses on several lime and clay-mineral mixtures. The lime began to attack the crystal lattices of the minerals immediately after compaction of the soil. The old compounds were destroyed and new ones were evolved.

The initial reaction between kaolinite, considered a stable mineral, and the lime occurred within the lattice and on the surfaces between the layers as well as around the edges of the structure. Some new crystalline products, probably hydrous calcium silicates, were formed at once. Eades and Grim (1960) believed that "several different compounds may exist at





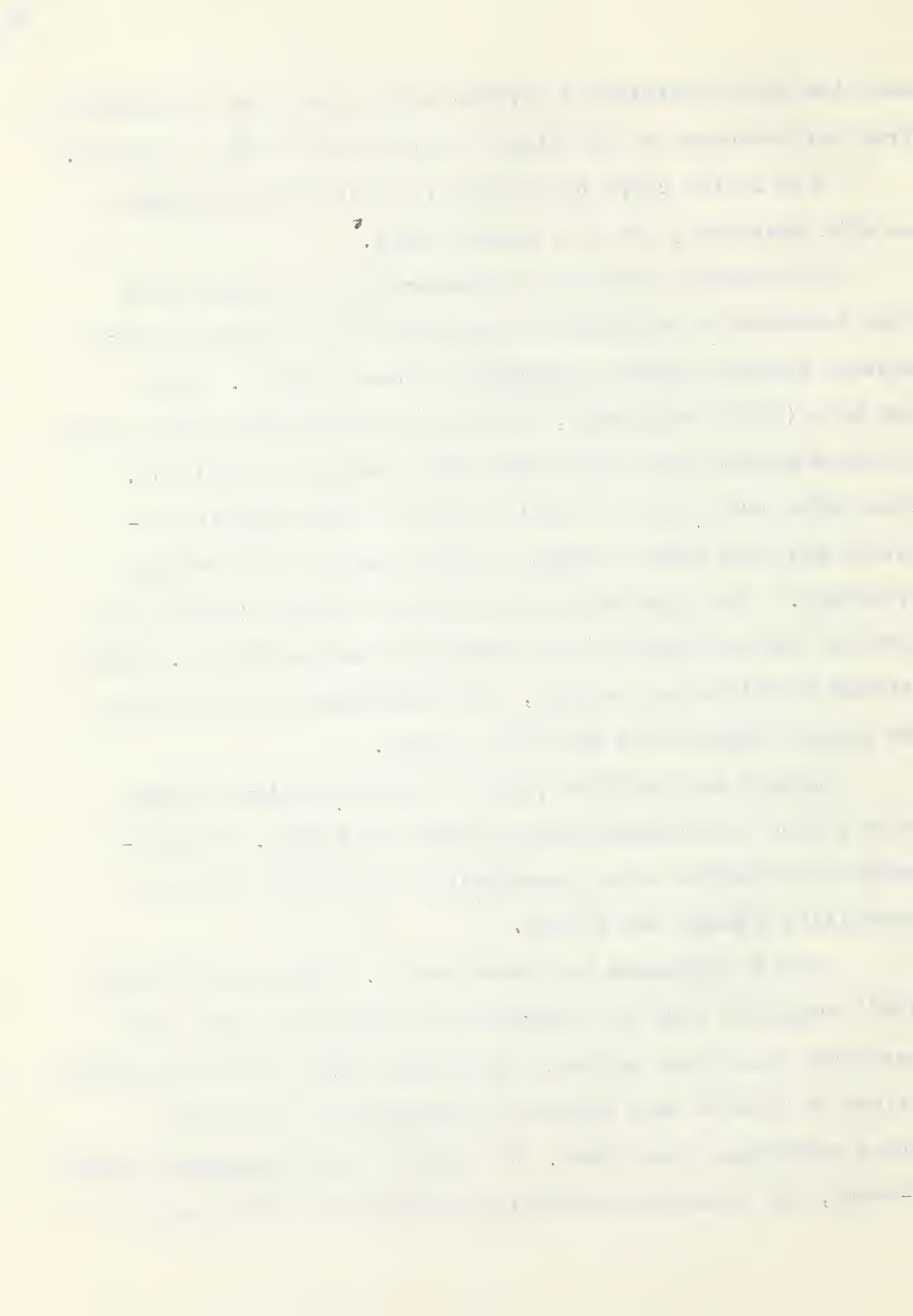
one time until equilibrium between the silica which is removed from the structure of the clays and the excess lime is reached."

For illite clays the reaction was in the same manner as with kaolinite, but at a reduced rate.

Diffraction patterns of montmorillonite treated with lime indicated a substantial breakdown of the original crystal lattice without apparent formation of new crystals. Eades and Grim (1960) postulated, "There is a possibility that calcium silicate hydrate gels are formed which are not crystalline. These gels could just as easily interlock the particles together and give added strength without having a crystalline structure." The specimens used by Eades and Grim (1960) were cured at 60C and dried in the absence of carbon dioxide. Since neither condition was natural, the conclusions may be invalid for normal temperatures and field curing.

Leonard and Davidson (1959) reported similar products being formed in reactions between lime and flyash. They deduced the formation of an intermediate gel before the final crystalline product was formed.

Calcium carbonate is a weak cement. Herrin and Mitchell (1960) suggested that the formation of carbonates might also discourage pozzolanic action. Taylor and Arman (1960) attributed failure of several base courses to carbonation of the soil before compaction took place. If compaction was completed before 48-hours, the pozzolanic cementing activity was sufficient to





overcome harmful effects of carbonation. However McDowell (1959) felt that once compaction had been completed and the pozzolanic reaction was proceeding, carbonation might give beneficial results.

#### II.4 VARIABLES WHICH AFFECT THE STRENGTH

Pavements must depend upon their undrained strength to resist deformation caused by the rapid application of wheel loads. An unconfined compression test on a saturated sample would indicate the minimum strength one would expect in a stabilized clay subbase. It was chosen for this investigation because it was a simple test which could be completed within a few minutes. The results of unconfined compression tests cannot be employed directly in the common empirical methods of pavement design. However, the results would indicate the mixes and the densities most promising for investigation by the CBR or stabilometer methods commonly used in practice.

The strengths obtained from unconfined compression tests could be subject to variables depending upon variations in (1) the constituents, (2) the stabilizing processes, and (3) the testing procedures.

##### Variations in Constituents

Type of lime. Lu and others (1957) conducted strength tests using dolomitic limes, synthesized from reagent grade chemicals, in mixtures with plastic soils. The specimens, in



which the lime content exceeded seven per cent of the dry weight of soil, displayed the highest strengths when dolomitic quicklime was used. The other limes, arranged in the order of decreasing strength, were monohydrated dolomitic lime, dihydrated dolomitic lime, hydrated calcitic lime and quicklime. The order of the last three limes was sometimes inverted for quantities of lime that were less than seven per cent of the dry weight of soil. Equal parts of calcium to magnesium, in the proportions of their molecular weights, gave the highest strengths. They found, when using commercial limes, that the values of strengths were reduced but the arrangement remained unchanged.

Type of pozzolan. Davidson and others (1958), after comparing four different samples of flyash, concluded that the reactivity was dependent upon the percentage of the flyash passing the number 325 sieve and upon the carbon content. Hollan and Marks (1960) doubted that a correlation existed between the amount of carbon and the reactivity of flyash.

It is questionable if these conclusions could be applied to pozzolans other than flyash, such as the calcined shale used in this investigation.

Type of soil. The pozzolanic activity of a soil determines its capacity for lime stabilization. Herrin and Mitchell (1960) reported that highly plastic clays generally developed larger increases in the 28-day strengths than did



clays of low plasticity or silts. They also mentioned two soils in India which were predominantly loam and gave decreased strength values when lime was added.

Quantity of additive and the lime-pozzolan ratio. The variability of the constituents discourages any attempt to draft a rational design for lime-pozzolan mixtures. Some authorities have published suggestions, based upon their experiences, with lime and pozzolan, which one might use as a guide for local investigations.

Clare and Cruchley (1957) presented tabulated values, copies from a Russian publication by Levchanovskii, of the optimum lime contents for satisfactory stabilization. Levchanovskii recommended using four to five per cent lime for sandy clays and mechanically stable soils; six to seven per cent for light and medium-textured silty and clayey soils; and seven to eight per cent for heavy silty and clayey soils.

Davidson and Handy (1960) presented the most favorable admixtures of lime and pozzolan derived from their experience. The amount of lime-pozzolan admixture varied from five to twelve per cent for clay soils. The amount of pozzolan (flyash), which was added, varied according to the pozzolanic activity of the clay.

Hollan and Marks (1960) suggested that "lime contents less than three per cent may result in a non-uniform mixture or may not develop satisfactory pozzolanic strength." To obtain





any significant increase in strength, the lime content should at least exceed the lime fixation capacity of the soil.

### Variations in the Stabilizing Process

Mixing time and rotting time. Taylor and Arman (1960) concluded that the failures in the soil-lime bases under their scrutiny were caused by lack of cementation and insufficient depth and width of pavement. The lack of cementation was attributed to poor mixing, due to the high clay content; and to delayed compaction of the soil-lime mixtures. They restricted the time between mixing and compacting, referred to as rotting time, to 24-hours unless fresh lime was mixed into to soil just prior to compacting the mixture. The rotting period allowed the lime to break up the clay clods permitting a more uniform and friable mix.

Mitchell and Hooper (1960) conducted some experiments to discover the effect of rotting upon the dry density and strength of stabilized clay-lime mixtures. A dolomitic hydrated lime was added in the amount of four per cent to an expansive clay of medium plasticity. Samples which had rotted for 24-hours before compaction had dry densities that were 8 pounds per cubic foot less than those of specimens compacted immediately after mixing. "Decreases in the strengths of cured specimens were observed for increased times between mixing and compaction." The abstract did not manifest whether these were immersed strengths





or strengths of partly saturated specimens.

Density. Herrin and Mitchell (1960) emphasized the importance of compaction in the strengths of lime-soil mixtures. The increased strength came from the higher densities produced by additional compactive effort and the strength was not related to changes in density produced by varying the percentage of additive.

Adams (1959) indicated a marked decreased in strength resulted if the mix is appreciably above or below the optimum strength moisture content. For the soils tested, he found the optimum strengths moisture content to be equal to, or slightly less than, the moisture content required to achieve optimum dry density.

The voids-cement ratio. Larnach (1960) postulated a unique relationship between the strength and the voids-cement ratio of a stabilized soil-cement mixture. His hypothesis is presented because it is believed that the same relationship may exist for some soil-lime mixtures. The voids-cement ratio was defined as the absolute volume of the air and water divided by the absolute volume of the cement. The equation for the calculation of the voids-cement ratio is derived in appendix E.

A series of tests was conducted by Larnach (1960) using a poorly graded clean fine sand mixed with a powdered clay fraction in the proportions of 9:1 by weight. Three different



contents of the same cement were tried. All of the specimens were compacted with the same compactive effort; the dry density being varied by changing the moisture content of the mixtures.

From this data he showed the following empirical relationship to exist:

$$S = \frac{5600}{\left(\frac{V}{C}\right)^{1.54}}$$

where S equalled the strength in pounds per square inch,

V equalled the volume of the voids,

and C equalled the volume of the cement.

It is emphasized that this relationship is given as an example only, since it was based on only one type of cement and a single compactive effort.

Time of curing. Herrin and Mitchell (1960) reported test results by McDowell showing an increase in strength over periods of two to four years duration, depending upon the lime content and other factors; after which, there was little gain or even a slight decrease in strength with further aging.

Dumbleton and Ross (1960) displayed the results of strength tests of ten different clays, sampled from various parts of the United Kingdom. Nine of the clays showed a substantial (twofold to threefold) increase in the unconfined compressive strength between the first week and the fifty-second week of normal curing (25C). Ten per cent lime was used to



stabilized all of the mixtures. No explanation was given for the failure of the tenth sample, a Weald clay, to develop strength after one week. Most of the stabilized soils showed 7-day strengths which were at least 75 per cent of the 28-day strengths. The study also showed that the 28-day strength was not an indicator of the strength after one year of curing at 25C.

Curing temperature. Lime-soil and lime-pozzolan-soil stabilization can be accelerated by the application of high temperatures. Dumbleton and Ross (1960) demonstrated that the strength of a silty clay and a heavy clay varied as the log of time for all temperatures up to 25C. For the heavy clay this relationship continued up to temperatures of 45C. For the silty clay, however, the rate of strength gain was much more rapid at 45C.

Although there was not an exact correlation between the 7-day strength of specimens cured at 60C and the 28-day strength cured at 20C, Hollan and Marks (1960) claimed the accelerated curing would give results that were indicative of the 28-day strength. Davidson and Handy (1960) offered a rebuttal to this argument, by saying "some mixes which react well at 140F have been found to be practically unreactive at ordinary temperatures."

Curing Moisture. Upon examination of reports which claimed a benefit in strength from moist curing, it was found





that comparative specimens had not been cured under dry conditions. McDowell and Moore (1948) asserted that dry curing had an advantage over moist curing, but the dry curing in their tests was at a temperature of 60C whereas the moist curing ensued at room temperature..

Definite proof as to whether alternate wetting and drying increases strength seemed to be lacking. Herrin and Mitchell (1960) summed up the controversy: "In some instances specimens cured at low humidity. In other cases higher strengths were developed in those specimens cured at high humidity. It does not influence strength to the same extent as variations in curing temperature."

### Variations in Testing Procedure

The undrained strength of a clay depends upon many factors, such as the moisture content at failure; the rate of stress application; and the size, shape and end conditions of the specimens. Other important variables which were not considered in this type of investigation are the preconsolidation load; and the direction and magnitude of the principle effective stresses. It was not known to what extent the addition of lime and pozzolan to the clay would affect any of these factors. With the possible exception of the moisture conditions at failure, all of the variables were maintained at certain values for all of the tests in this investigation. Thus, they





will not affect the internal comparison of the results. In comparing the results from other research projects with this study, however, the manner of testing cannot be ignored.

Moisture conditions at failure. A partly saturated cohesive soil, untreated with lime or pozzolan, will lose most of its strength if it becomes saturated. Therefore, the most severe test for a lime treated soil would probably be a strength test on a saturated specimen.

If two untreated clay specimens are both saturated, the one with the lower water content will exhibit the higher strength. The same relationship may also be true for treated specimens, which have the same lime-pozzolan content; and which has experienced identical procedures of mixing, rotting, curing and soaking. When comparing two specimens which have been treated differently, the pozzolanic cementing action undoubtedly is such a dominating influence on the strength that the effect of the water content at failure becomes insignificant.

Duration of test. Tests of short duration give the highest undrained strength. Whitman (1960) has suggested that the strengths of soils which exhibit a brittle type of failure are more susceptible to variations in the duration of the test than are the strength tests on plastic soils. Since lime causes a soil to become more friable, the rate of application of stress would be an important factor in the determination of the ultimate strength of lime-stabilized specimens.



End conditions of the specimens. For research work, the testing machine should have an adjustable bearing block. Withey and Aston (1925) stated that placing the bearing block on the table was equally as effective as hanging it from the head of the testing machine. The specimen, bearing block and testing machine should have a common axis.

Withey and Aston (1925) also recommended that use of bedments to ensure that the contact surfaces of the specimens were plane and parallel to the surfaces of the head and bearing block. The bedments, which they recommended, were to be as thin as possible, composed of a material which would not spread or flow under pressure, and which possessed the same strength and elastic properties as the specimen.

In this investigation a bearing block, set on the moving table, was used for the strength tests, but the specimens were broken without employing bedments. Bedments were not employed because the gain in strength attributed to the bedments could also have been caused by the drainage of water from the specimen during the time required for the bedment to set. Since the strengths would be lower without the use of bedments, the error in the strength, if it existed, was on the conservative side.

Size of the specimen. "For materials which rupture on planes inclined at more than  $45^{\circ}$  with the horizontal, specimens of equal height and diameter are not suitable since the strength is increased by frictional resistance acting on the surfaces under pressure." The quotation came from Withey and Aston (1925).



For such materials, they recommended specimens with a height to diameter ratio of two. A correlation between the strengths of cylindrical specimens that have different height to diameter ratios may be performed by enacting an exhaustive testing program on the particular material, since the relationship varies with each material. Apparently such a program has not been attempted for lime-pozzolan samples.

## II.5 DURABILITY OF CLAY-LIME-POZZOLAN STABILIZATION

Authorities do not agree on the constitution of a standard of measurement for the durability of lime stabilization. The ability of a soil to withstand alternate periods of freezing and thawing is generally agreed to be the most severe criterion of durability for soils treated with lime. Other tests have been designed to measure the ability of the specimen to withstand cyclic wetting and drying or cyclic heating and cooling. Although some authorities, such as Davidson and Handy (1960), claimed that wetting and drying might even be beneficial to a lime stabilized soil, there is no proof that soils in Alberta would react in this manner. Therefore lime treated soils, which have successfully passed freeze-thaw tests, should be subjected to a wetting-and-drying test and possibly to a heating-and-cooling test before they are pronounced durable.

A good laboratory test should give results that are duplicated in field trials. Several laboratory freeze-thaw tests have been proposed, but the point of failure of the field





installation may be the same or it may be lower than the point of failure as shown by the laboratory test. The factor of safety against a failure in durability is unknown, and the question which asks, "What constitutes the point of failure?" has not been clearly answered.

This section of the report will attempt to interest the reader in successful field installations and follow up with a brief description of some laboratory freeze-thaw tests. An analysis of the advantages and disadvantages of the tests would be unfair since none of them were tried in this investigation.

#### Cases Reported By The National Lime Association

Some of the earlier successes of lime stabilization were discussed by the National Lime Association (1954). The first uses of lime were in reconstructing old bases, using small (three per cent) amount of lime. Then some of the highway maintenance forces of Texas began to mix approximately three per cent lime by weight of the dry material into bases of caliche-clay, river gravel, marl-gumbo and clay-gravel. In 1953, in Jefferson County, Texas, the upper 8 inches of a clay hydraulic fill subgrade was treated with four per cent lime. A base, 18 inches thick, was placed above this subgrade.

#### Experience With Lime Stabilization In The State of Texas

McDowell (1959) prepared a report to discuss desirable testing and construction techniques for successful lime stabilization. He attributed early failures of roads stabilized





with lime to lack of close control of the mixing, compacting, and curing of the treated material. In addition, the earlier roads, which were built before 1945, were left unsealed and they deteriorated rapidly under traffic. To prove these assertions, McDowell (1959) cited the case of a road which had been successfully stabilized using the same percentage of lime, and the same materials that had been used in one of the early failures. However, McDowell (1959) warned the reader that "due to the effects of weathering conditions, clay-lime mixtures which are adequate for subbase purposes probably will not prove to be satisfactory if used as a final course of base upon which a thin surfacing is to be placed."

Some roads in Texas were still in excellent condition, 14 years after being stabilized with lime. The maximum amount of lime used in the State of Texas for the stabilization of a clay was 8 per cent of the dry weight of the clay soil. One should remember, however, that damage by frost is not prevalent in that region.

McDowell (1959) also indicated the anomaly that "the mixes which are hardest at an early age are not necessarily the best because some of the differences in strength may be due only to the rate of setting being speeded up."

Although he found that the field strengths of lime-granular mixes usually were higher than the laboratory strengths, McDowell (1959) found the converse to be true for clay-lime mixtures due to delayed mixing, compacting, and weathering.



### Experience With Lime Stabilization In The State of Missouri

Jones (1958) reported the results of a test section of an old base course which was improved by the addition of  $4\frac{1}{2}$  per cent lime. The old pavement consisted of only  $\frac{1}{2}$  inch of bituminous surface over 2 inches of gravel base. In the reconstruction, this material was mixed with the subgrades of clays, silty clays, and silty clay loams to form a new base 7 inches in depth. Quicklime, hydrated lime, a combination of both quicklime and hydrated lime, and a waste lime were used in various sections of the highway. The road surface consisted of a light bituminous seal coat.

After  $4\frac{1}{2}$  years of service, all of the test sections were in good condition with the exception of the portion stabilized with the waste lime. The sections stabilized with a combination of hydrated lime and quicklime displayed the most hardening. However, the most durable section, that is, the section which required the least maintenance, was the portion stabilized with hydrated lime only. In that portion, which was 1300 feet in length, the total repairs consisted of two small patches, each 16 square feet in area, and an edge patch. The edge patch, however, was attributed to insufficient thickness of the base at the edge of the pavement.

Two facts were illustrated which should be repeated for emphasis. The seal coat could be reduced in thickness until it was just sufficient to prevent the air from contacting the upper surface of the base. The most durable pavement was not



necessarily the hardest material.

The contradiction with McDowell's (1959) opinion concerning the use of a lime-stabilized clay as a base course, should be noted. Perhaps Jones considered the roads were in good condition, while McDowell would consider the same conditions inadequate. The inclusion of the argument in this report is academic because none of the roads in either State were subject to the durability conditions caused by severe freezing and thawing cycles.

#### Lime and Lime-Pozzolan Stabilization In Canada

The first major Canadian project which utilized a lime-stabilized subbase was a section, 1.6 miles in length, of the Selkirk By-pass, a 32 foot wide provincial highway in Manitoba. The thickness of the subbase was alternately 6 or 8 inches. The 8 inch thickness consisted of clay stabilized with either 3, 5, or 7 per cent of slurried quicklime. The 6 inch sections received either 4, 6, or 8 per cent of the quicklime slurry. A rotting period of 2 days was allowed before the mixture was compacted. The base consisted of 6 inches of imported material with a clay binder. The wearing surface was a  $2\frac{1}{2}$  inch bituminous mat. Further details may be obtained in a paper by Robertson (1959).

The subbase was stabilized in June, 1959, and Hearn (1961) reported in the Heavy Construction News that the project had been completed. Hearn (1961) reported that the Materials Testing Branch of the Manitoba Department of Public Works had completed





one study of the system and they were planning to conduct a second study before making a conclusive report.

In 1961 a contract was let to stabilize some shoulders on the Trans-Canada Highway, just west of Winnipeg, with lime.

The Municipality of Assiniboia, on the outskirts of Winnipeg, used lime-stabilization in their 1961 road building program. An 8 inch clay-lime stabilized base was covered with 1 inch of asphalt. The lime was approximately five per cent of the dry weight of the heavy clay, which has the local name of gumbo. The lime and clay were allowed to rot for 2 days before the final mixing and compaction was permitted.

A test strip was laid in Assiniboia in August, 1960, and was left unsurfaced throughout the winter of 1960-61. According to Hearn (1961) the results of the test were favorable.

Adams (1959) reported favorably on some field installations in which lime and flyash were used to stabilize sand, gravels, and silty sands. However, at the time of Adam's report, the Hydro-Electric Power Commission of Ontario had not laid any test strips using a clay soil.

#### Current Comment on Durability From the United States of America

Herrin and Mitchell (1960) suggested the following factors influenced the durability of lime stabilized bases and subbases.

They contended that higher lime contents were more resistant to deterioration than mixtures with smaller amounts of lime. The durability increased with curing time and with higher



densities when those densities were caused by increased compactive effort and not by variations in the content of lime.

Davidson and Handy (1960) observed that the total amount of lime and pozzolan additive might have to be increased to 20 or 30 per cent to successfully resist the effects of freezing and thawing in a clay soil.

Davidson and Handy (1960) also suggested that mixtures prepared from monohydrate dolomitic lime were more resistant to freeze-thaw than mixtures prepared from other types of lime.

#### Means of Determining the Durability of a Soil-Lime-Pozzolan Mixture

The most widely used method of determining the durability of lime-soil and lime-soil-pozzolan mixtures was the method recommended by the American Society for Testing Materials (1958) for the evaluation of the durability of soil-cement specimens. The compacted specimens were placed on a saturated felt pad and subjected to cyclic freezing and thawing. After each thawing cycle the specimens were brushed with a wire brush and the loss in weight was recorded. The loss in weight required to reject a specimen was not specified.

The modified British standard test used in the investigations at Iowa, placed a specimen in the opening of a wide-mouthed thermos bottle filled with sufficient water so that an open-system of freezing was allowed. The frost entered the



specimen from the top only. The specimen was given several cycles of freezing and thawing and was then broken to discover its compressive strength. If the compressive strength was less than eighty per cent of the strength of a companion sample, which had been soaked in water, the specimen was considered a failure. In addition, a minimum strength was also expected of the freeze-thaw sample.

Minnick and Williams (1954) introduced a third type of test. Specimens were compacted into 6 inch or 12 inch cubes and set on a sample of untreated subbase, which was several inches thick. In some cases the subbase material was also packed around the sides of the specimens. "Thermistors", presumably thermo-couples, were imbedded in the specimens at depths of 2 and 5 inches below the surface. The complete assembly was submerged in water for 24 hours, after which it was removed and a block of dry ice was set on the upper surface of the specimen. The dry ice was taken away when the lower thermistor indicated frozen conditions at that depth. The freeze-thaw cycle was repeated ten times, with the assemblage being "resaturated" after each thaw period. Changes in the internal structure were observed by timing the rate of advance of the freezing temperature between the thermistors. Another criterion for durability was the amount of heave experienced by the specimens during the freezing part of the cycle.

Each method of freeze-thaw testing has its advocates and its critics. The method chosen to measure the durability



of the lime-soil mixture in this investigation was an adaption from the modified British test.





### CHAPTER III

#### TRACE CHEMICALS USED AS ACCELERATORS

Chemicals have been used in quantities such as  $\frac{1}{2}$  and 1 per cent by weight of the dry mixtures to increase the rate of strength development of stabilized soil-lime and soil-lime-pozzolan mixtures. Since the durability of lime-stabilized soils is primarily a function of the strength, any accelerating process which would yield a higher early strength would also be a factor in preventing the destruction by frost action of a lime-stabilized subbase during the first winter of service. The reduced interval required for strength development between the time of compaction and the winter season, also permits a longer construction season.

Davidson and others (1959) conducted a series of experiments using 47 chemicals with Ottawa sand-lime-flyash mixtures. The highest increases in the 28-day strength were shown by the mixtures treated with bases or basic salts. Sodium hydroxide and sodium carbonate, in particular, gave high early strengths when used in the amount of  $\frac{1}{2}$  per cent. Some neutral salts such as potassium permanganate and other strong oxidizing agents also produced good results when used in the same quantities.

Sodium hydroxide, sodium carbonate, potassium permanganate and calcium lignosulphonate were chosen for investigation as accelerators for the clay-lime and clay-lime-pozzolan reactions in the present study. The first three chemicals were chosen



because of their favorable record when used with sand and flyash. The calcium lignosulphonate was studied to discover if the stabilizing influence and the ability to resist frost action possessed by the calcium lignosulphonate mixtures would reinforce the strength and durability characteristics of the lime-clay pozzolanic reaction. This review of the published literature was restricted to the four chemicals just mentioned. However, the chemicals are representative of a large number of bases, basic salts, oxidizing agents and organic dispersants which could have been used as additives to the lime-soil mixtures.

### III.1 SODIUM HYDROXIDE AND SODIUM CARBONATE

Davidson and others (1959) offered an explanation for the beneficial effects caused by the addition of a strong base to a mixture experiencing pozzolanic cementation. The sodium hydroxide, for example, would probably react with the siliceous material from the surface of the soil particles to form soluble sodium silicate, which in turn would interchange cations with the lime to form cementitious insoluble calcium silicate and sodium hydroxide. The reformed sodium hydroxide would then be available for further attack on the exposed silicates.

A basic salt, such as sodium carbonate, probably reacted with the lime to form sodium hydroxide and calcium carbonate. The cementing action of the calcium carbonate gave an increase in strength to the stabilized mixture. However, the formation of insoluble calcium carbonate reduced the concentration of



calcium ions available for combination with the silicate anions. Thus a mixture employing sodium carbonate as an accelerator would require a higher quantity of lime additive in order to achieve the same strength as a mixture using sodium hydroxide.

Ladd and others (1960) offered an explanation which coincided with that proposed by Davidson and others (1960), but, in addition, the following points were expounded. The introduction of sodium hydroxide or a sodium salt to a lime-soil mixture would increase the pH of the pore fluid. The high pH value would aid in the solubility of the silicates contained in the soil. The presence of sodium ions in solution would decrease the solubility of the calcium and reduce the concentration of calcium ions and so would slow down the formation of the cementitious calcium silicates. Thus, the sodium base or salt provided more uniform distribution of the silicates throughout the soil before precipitation occurred.

#### Results from the Use of Sodium Carbonate

Davidson and others (1960) introduced sodium carbonate into three different stabilized mixtures of montmorillonitic clay and calcitic hydrated lime. The chemical was added in amounts from  $\frac{1}{4}$  per cent to 4 per cent by weight of the dry mixture to the moulding water. The mixture was compacted immediately after the completion of a short mixing period. In all other respects, the mixing, compacting, curing and testing of the specimens followed a procedure which closely resembled that outlined in this study.





One of the soils, referred to as Kansan gumbotil, was noncalcareous and contained approximately 70 per cent clay sizes.<sup>1</sup> The other soils, called a plastic loess and a Kansan till, each contained 39 per cent clay sizes, but the non-clay sizes of the plastic loess were all silt sizes whereas the Kansan till contained equal fractions of silt and sand. The Kansan till was rich in calcium while the plastic loess was non-calcareous.

When the sodium carbonate was added to the Kansan till mixtures containing four or six per cent lime, all of the 28-day strengths were less than the untreated strength. Sodium carbonate added, in the amount of two per cent by weight of the dry mixture, to the Kansan till plus eight per cent lime, slightly increased the 28-day strength. Mixtures containing more or less than this amount showed reduced strengths when compared with the mixture that received no accelerator.

When the sodium carbonate was used to treat the plastic loess plus four per cent lime the results were negative for all concentrations from  $\frac{1}{4}$  per cent up to 4 per cent. In the series of mixtures containing 6 per cent lime, at least 2 per cent sodium carbonate was required to increase the strength at

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<sup>1</sup>The divide between sand and silt sizes was 0.074 millimeters, and between silt and clay sizes was 0.005 millimeters.



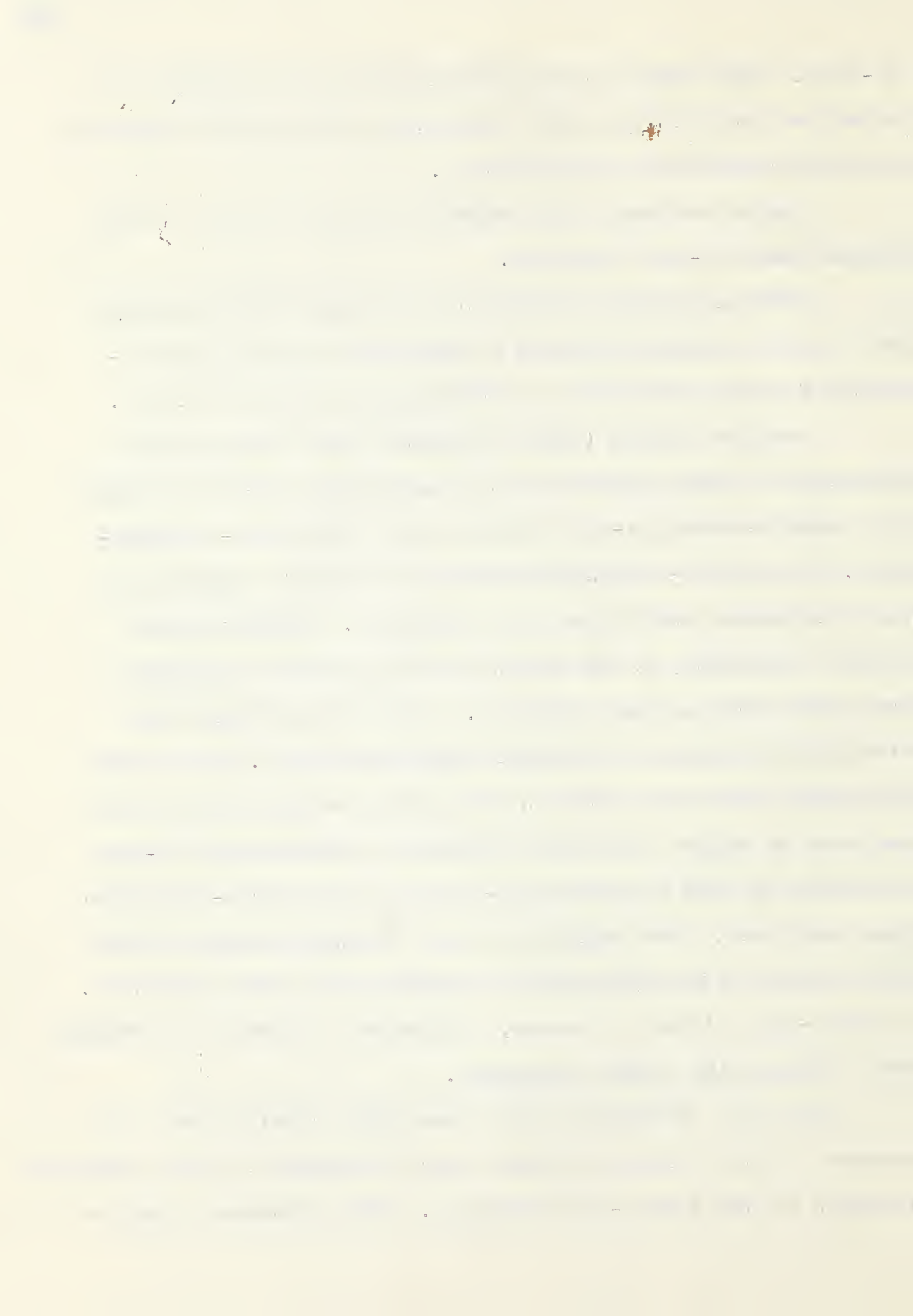
28-days. Only when the lime concentration was increased to eight per cent did the sodium carbonate give positive increases in every compressive strength test.

Sodium carbonate gave negative results for all of the Kansan gumbotil-lime mixtures.

From the results, Davidson and others (1960) concluded that sodium carbonate was not a predictable strength accelerator for clays which were predominantly montmorillonitic.

Ladd and others (1960) conducted tests using sodium carbonate in trace quantities to increase the 28-day strengths of a quartz-bentonite-lime mixture and a kaolinite-lime mixture. The quartz-bentonite mixture of minerals comprised 90 per cent quartz and 10 per cent bentonite. Reagent grade calcium hydroxide, in the amount of ten per cent by weight, was mixed with the pure minerals. The test specimens were statically compacted in Harvard miniature moulds. The sodium carbonate, which was added in the amounts varying from  $\frac{1}{2}$  to 2 per cent by weight of the dry mixtures, increased the 28-day strengths of both the kaolinite-lime and the quartz-bentonite-lime specimens. The magnitude of the strength increase varied considerably with changes in the quantity of sodium carbonate. A quartz-lime mixture, however, displayed a decrease in strength when treated with sodium carbonate.

From the information given one might conclude that the presence of the bentonite mineral was the reason for the increased strength of the quartz-lime mixtures. This statement seems to



contradict the conclusions of Davidson and others (1960) concerning the advantages derived from using sodium carbonate with a montmorillonite. In the tests conducted by Ladd and others (1960) the specimens containing sodium carbonate were compacted to the same dry density as the specimens which received no accelerator. The specimens produced by Davidson and others (1960) all received the same compactive effort and thus the dry density varied between the specimens treated with sodium carbonate and those left untreated. Thus the compacted dry density could have been a cause for the conflicting evidence. There were also differences between the amounts and types of lime and probable differences between the montmorillonites.

#### Results From the Use of Sodium Hydroxide

Davidson and others (1960) tried adding sodium hydroxide, in the amounts used in the sodium carbonate experiments, to the same soil and lime mixtures. They found that sodium hydroxide increased the 28-day strengths of the Kansan till mixtures containing four, six and eight per cent lime provided the sodium hydroxide content was at least one per cent. The plastic loess-lime and the Kansan gumbotil-lime stabilized mixtures could achieve higher 28-day strengths if the sodium hydroxide was added in the amounts of two or three per cent. When lesser quantities of sodium hydroxide were used the mixtures either displayed a slight gain or a small decrease in strength.

Ladd and others (1960) substituted sodium hydroxide for





sodium carbonate in their experiments also. They found that the addition of sodium hydroxide increased the 28-day strengths of all the quartz-lime, quartz-bentonite-lime and kaolinite-lime mixtures which were tested. The lime content in these tests was ten per cent of the dry weight of the minerals.

They also employed sodium hydroxide as a strength accelerator for a stabilized mixture of Vicksburg buckshot clay and five per cent commercial calcitic hydrated lime additive. The inorganic clay comprised 35 per cent clay sizes and 65 per cent silt sizes.<sup>1</sup> The liquid limit was 60 per cent and the plasticity index was 35 per cent. The clay had a cation exchange capacity of 30 milliequivalents of hydrogen per 100 grams of dry soil. About equal parts of montmorillonite and illite were indicated by an x-ray diffraction analysis but there was no evidence to indicate the presence of kaolinite. The effect of the addition of sodium hydroxide to the 28-day strength of the stabilized lime-clay mixture is presented in tabular form:

Approximate Concentration of NaOH % of Dry Weight of Mixture	Approximate Immersed Compressed Strength p.s.i.
0	200
0.34	282
0.65	290
1.31	250

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<sup>1</sup>The divide between clay and silt sizes was 0.002 millimeters and between silt and sand sizes was 0.074 millimeters.





When mixed with lime and an organic clay, however, sodium hydroxide failed to increase the strength and in one case it even caused a reduction in strength. Ladd and others (1960) explained the reaction in this manner:

In an organic soil-lime system, both the soil silica and the organic matter are competing for calcium ions. In soils with large amounts of organic matter, the organic compounds can progressively extract calcium ions from any weakly bonded calcium silicates which results in a reduction in strength. Moreover, the addition of most alkali additives not only solubilizes the soil silicates, but also intensifies the organic activity.

### The Effect of Rotting Time on Strength

The results discussed in the chapter up to this point have all been based upon specimens which were compacted immediately after mixing. The procedure followed in the present study allowed a rotting period of 24 hours between the times of mixing and compaction. A brief discussion of the effects of rotting upon stabilized mixtures treated with sodium silicates is presented. Since sodium silicate was a reaction product when sodium hydroxide and sodium carbonate were added to lime-soil mixtures, the discussion is pertinent to this review.

Ruff and Davidson (1960) studied the effect of rotting on the dry density, compressive strength, and optimum moulding moisture content of a montmorillonitic clay soil stabilized with six per cent calcitic hydrated lime and five different powdered sodium silicates. They found that (1) as the rotting time increased, the dry density and the compressive strength decreased;



(2) the strength was not correlated with the dry density since the rotting time affected the optimum moulding moisture content both for maximum dry density and for maximum strength, but the effect was not the same. The abstract did not distinguish between the decreases in strength and density that could be attributed to the lime and the changes caused by the sodium silicate.

Durability of Mixtures, Accelerated by the Addition of Sodium Hydroxide and Sodium Carbonate

Davidson and others (1960) conducted some modified British freeze-thaw tests on the Kansan till stabilized with six per cent calcitic hydrated lime. They also ran similar tests on mixtures of the same materials with  $2\frac{1}{4}$  per cent sodium carbonate added and with one per cent sodium hydroxide added. The results of specimens cured for 28-days at 22C are presented in tabular form:

Type of Accelerator % of Dry Weight of Mixture.	Strength of Freeze-Thaw Specimen. p.s.i.	Strength of Freeze- Thaw Control Specimen p.s.i.
none	103	122
Na <sub>2</sub> CO <sub>3</sub>	322	418
NaOH	341	420

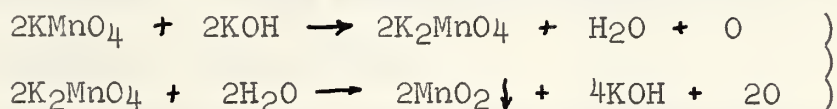
### III.2 POTASSIUM PERMANGANATE

Davidson and others (1959) showed that potassium permanganate gave approximately the same 28-day strengths as sodium hydroxide when introduced to accelerate the stabilization of



Ottawa sand-lime-flyash mixtures. Examples seemed to be lacking in the published literature regarding the use of potassium permanganate as an accelerator of clay-lime stabilization.

Thorpe's Dictionary of Applied Chemistry (1946) contained the following equations, representing the reactions which take place when potassium permanganate is added to an alkaline solution.



Thus  $1\frac{1}{2}$  molecular weights of oxygen atoms were produced from each molecular weight of potassium permanganate. Davidson and others (1959) believed that the carbon present in the flyash was oxidized, with the subsequent formation of potassium carbonate. The potassium carbonate in reactions with calcium hydroxide would form cementitious calcium carbonate and potassium hydroxide.

The plentiful supply of potassium hydroxide would then benefit the strength in the manner that has already been described for sodium hydroxide. Davidson and others (1959) also believed that the oxidation of the carbon in the flyash and the cleaning action of the potassium permanganate on the surfaces of the soils and flyash were a factor in increasing the strengths of the pozzolan mixtures.

This hypothesis, although based upon studies with sand and flyash, could likely be applied to clays and pozzolans other than flyash.







### III.3 CALCIUM LIGNOSULPHONATE

The commercial production of paper pulp depends upon the conversion of lignin into water-soluble sulphonic acids by heating with aqueous acid sulphites. The ligninsulphonic acids may be precipitated by the addition of calcium chloride. The resultant waste product is sold commercially in Canada under the brand name of Lignosol and is marketed in either liquid or powder form. It has successfully been used to prevent frost heaving in silty soils.

The resistance to frost action caused by the treatment of a soil with lignosol was attributed to the increased viscosity of the pore fluid by Hardy (1953) and to the dispersing properties of the chemical by Lambe (1956). Both explanations indicated that the lignosol functioned to retard the movement of water to the frost line. Penner and Robillard (1960) suggested that an additional mechanism might affect the reduction of the frost heave. They suggested that the addition of lignosol to the soil reduced the rate of crystal growth and thus the segregation of ice at the frost line was reduced.

Waste calcium lignosulphonate has also been tried as a constituent to stabilize soils. Nichols and Davidson (1958) described the possibilities of combining lignin with such heavy metals as chromium, iron, cobalt, copper, and mercury to form stable water insoluble complexes. The lignin would bond the adjacent soil particles together by linking to the metal cations



at the mineral surfaces. Lignosulphonate becomes an anion in solution and thus would not be attracted to the negatively charge soil particles.

The addition of calcium lignosulphonate to a soil-lime stabilized mixture might weaken the strength since the dispersant properties of the lignin would counteract the aggregating properties of the lime.



## CHAPTER IV

### THE TEST PROGRAM

The first section is devoted to a description of the materials used in the investigation. The program of the investigation is outlined in the second section. The third describes the complete procedures of the program, and the following section describes the reasons for adopting these procedures. The last section analyses the reliability of the results.

#### IV.1 DESCRIPTION OF CONSTITUENTS

##### The Soil

The soil was a highly plastic clay from a borrow pit, 200 feet to the right of station 419 + 00, highway 2-G-3, and from a depth of six feet; located near the town of Fahler, Alberta. Clear crystals, believed to be gypsum, were visible in the dry soil, but they were not evident when the moisture content was near the plastic limit. The clay was air-dried, then pulverized and screened through a number 10 sieve.<sup>1</sup> The sifted sample was thoroughly mixed, then sacked, and stored in a dry room at normal temperatures. The average moisture content of the air-dried soil was 4 3/4 per cent.

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<sup>1</sup>U. S. Standard series.



X-ray diffraction analysis. The results of an x-ray diffraction analysis performed by the Research Council of Alberta are displayed in tabular form.<sup>1</sup>

<u>Clay Material</u>	<u>Per Cent</u>
Illite	53
Kaolinite	26
Montmorillonite	21

Flame photometer analysis.<sup>2</sup> The distilled cation exchange capacity of the clay soil was 28.8 milliequivalents per 100 grams of air-dried soil. The total cation exchange capacity as determined by the flame photometer method was 74.5 m.e./100 grams of air-dried soil. The ions, both adsorbed and in solution were:

<u>Ions</u>	<u>Per Cent</u>
Ca <sup>++</sup>	68.5
Mg <sup>++</sup>	20.5
K <sup>+</sup>	9.5
Na <sup>+</sup>	1.5

Specific gravity. The specific gravity of the clay, using procedures outlined in Appendix A was 2.78.

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<sup>1</sup>The percentages given may be in error by the amount of 15 to 20 per cent.

<sup>2</sup>Analysis was made by Mr. S. Thomson, now Assistant Professor of Civil Engineering, University of Alberta, using the Alberta Soil Survey procedure.





Atterberg limits. The atterberg limits were determined using the procedures outlined in appendix A. The results were:

Liquid limit	77.5
Plastic limit	26.0
Plasticity index	51.5
Shrinkage limit	14.6

Grain size analysis. A grain-size analysis of the soil was made in accordance with procedures outline in appendix A. The results of the analysis were:

<u>Material</u>	<u>Limiting Diameters</u>	<u>Per Cent</u>
Sand sizes	greater than 0.06 mm.	8
Silt sizes	0.002 mm. to 0.06 mm.	30
Clay sizes	less than 0.002 mm.	62

The soil was classified as a highly plastic clay (CH) under the Unified Soil Classification.

### The Pozzolan

The pozzolan was a red-colored calcined shale which was processed from a coal mine dump located at Diamond City, Alberta. The pozzolan was received in a 50-pound double-layered paper sack and was stored in the same room as the soil. Hvozanski used this same sack of pozzolan for his experiments.

The moisture content of the pozzolan was 0.75 per cent and the specific gravity was 2.73. Approximately 90 per cent of the pozzolan was finer than a number 325 sieve. The pozzolan



belonged to the montmorillonite type of activated shales.

A slight amount of dark grey substance, possibly carbon, floated on the surface of the water whenever the pozzolan was immersed in water.

The composition of the pozzolan, known commercially as Diamond City Shale was given in a private communication to the Research Council of Alberta.

<u>Compound</u>	<u>Per Cent</u>
Silica	61.98
Alumina	20.41
Iron Oxide	4.91
Calcium Oxide	2.60
Magnesium Oxide	2.50
Sodium Oxide	0.68
Potassium Oxide	0.12
Undetermined	4.08
Ignition loss	<u>2.82</u>
	100.10

### The Lime

A commercial hydrated calcitic lime from Loders Lime (Alberta) Limited was supplied for the tests in two double layered paper sacks. The lime was transferred into half-gallon jars equipped with screw-tight lids. Every jar received an equal portion from each sack. A third sack from the shipment was returned to the manufacturer for a chemical



analysis, which is included in tabular form:

<u>Compound</u>	<u>Per Cent</u>
CaO	74.8
MgO	1.2
FeO <sub>2</sub> , AlO <sub>2</sub>	0.4
SiO <sub>2</sub>	0.3
Ignition Loss	<u>22.9</u>
	99.6

The glass jars were packed full, to reduce the voids, and thus diminish the possibility of carbonation of the lime. The specific gravity of the lime was 2.76, as determined by the procedure outlined appendix A except that kerosene replaced water as the displaced fluid. The moisture content of the lime was 0.27 per cent.<sup>1</sup>

#### The Water

Distilled water was employed throughout the program.

#### The Accelerators

Sodium Carbonate. The sodium carbonate was supplied as monohydrate crystals, Na<sub>2</sub>CO<sub>3</sub>.H<sub>2</sub>O, reagent grade, manufactured by J. T. Baker Chemical Co., Philipsburg, New Jersey.

Sodium hydroxide. The sodium hydroxide was in the form of electrolytic pellets; and was termed Fisher certified reagent,

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<sup>1</sup>Total moisture driven off at temperatures of 115C.





catalog S-318, from Fisher Scientific Company, manufacturing chemists, Fairlawn, New Jersey.

Potassium permanganate. The potassium permanganate was a Fisher certified reagent, catalog no. P-279, from Fisher Scientific Company, Manufacturing Chemists, Fair Lawn, New Jersey.

Calcium Lignosulphonate. The calcium lignosulphonate was a commercial product "Lignosol", manufactured by the Lignosol Chemicals Limited and supplied by the Department of Highways, Province of Alberta.

#### IV.2 OUTLINE OF PROGRAM

##### For Strength and Durability Using Lime-Pozzolan Mixtures.

This part of the program was divided objectively into two portions. One series of test investigated the strength of clay-lime and clay-lime-pozzolan mixtures after completion of 28-days of moist curing and 1 day of soaking. The second series of tests was conducted to discover the loss in strength which clay-lime and clay-lime-pozzolan specimens experienced during 6 or 12 freeze-thaw cycles. Both series of tests were run concurrently and were derived from the same mixes. Thus the strength and freeze-thaw specimens were identical at the end of the curing period.

The additive contents, that is, the sum of the dry weights of the lime and pozzolan, which were used in the



experiments were 5, 10, 15 and 20 per cent of the dry weight of the clay.

The ratios, by weight, of the lime to pozzolan tried in the investigation were 10:0; 9:1; 7:3; 5:5; 3:7; and 1:9.

Specimens of equal additive content which possessed the same lime to pozzolan proportions were varied in density by changing the compactive effort used to form the specimens. Two densities were employed in the program. The lesser density attempted to achieve the same unit weight as would be obtained from a standard proctor test.<sup>1</sup> The higher density achieved only 95 per cent of the value obtained from a modified AASHO test.<sup>2</sup>

No specimens were produced with a lime content of less than two per cent. Six specimens were made for each strength test and twelve specimens for each freeze-thaw test.

The program examined 21 different mixes for each density. The mixes were numbered as shown on Figures 1 and 3. The specimens compacted at the lesser density and higher density were designated by the letters A and B respectively. The specimens used to determine the 28-day strength were denoted by the numbers 21 to 26 inclusive. Specimens marked 41 to 46

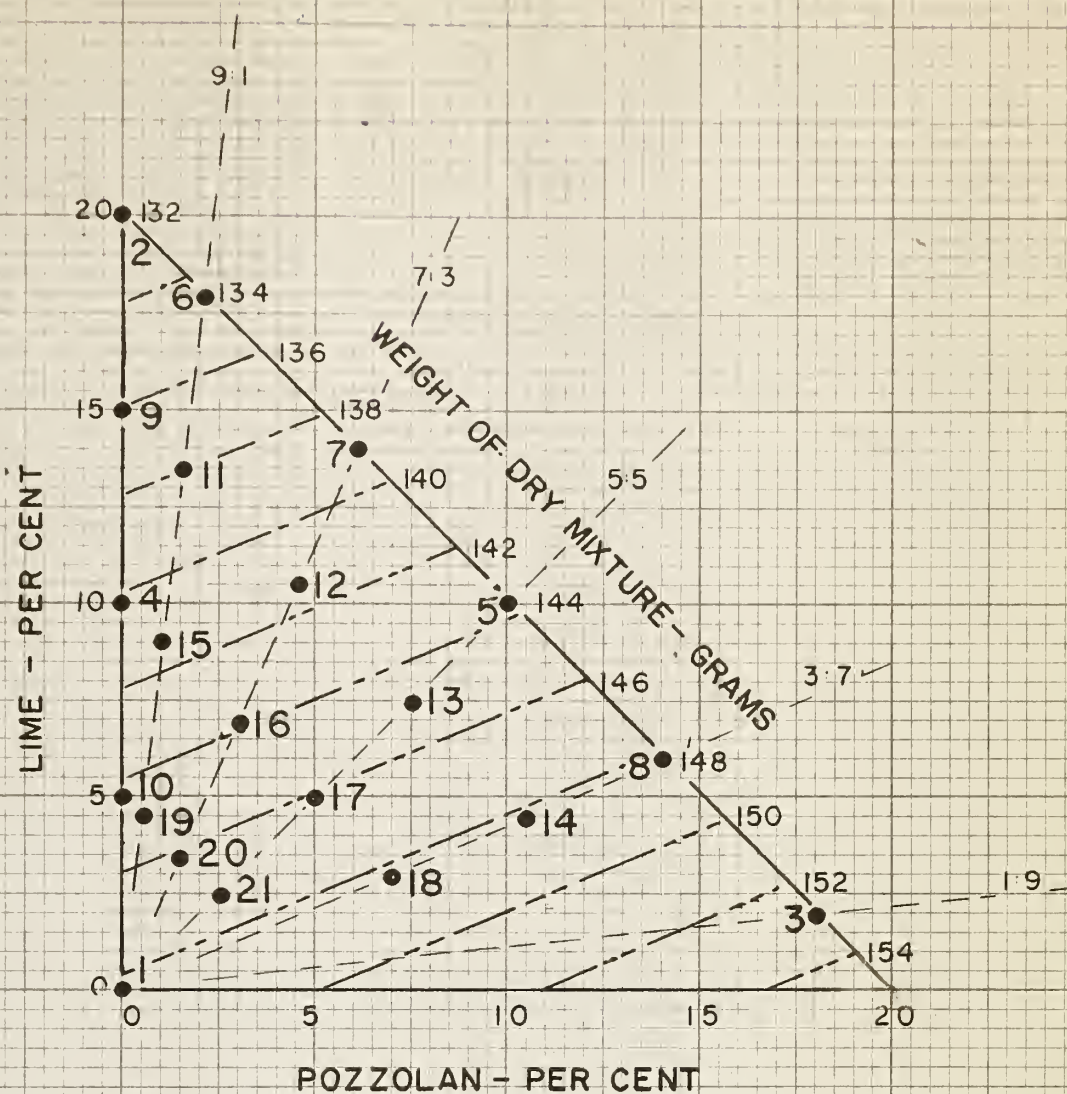
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<sup>1</sup>ASTM Designation D 698-57T, Method A.

<sup>2</sup>ASTM Proposed method A of Test for Moisture-Density Relations of Soils Using a 10-lb. Rammer and an 18 inch Drop.





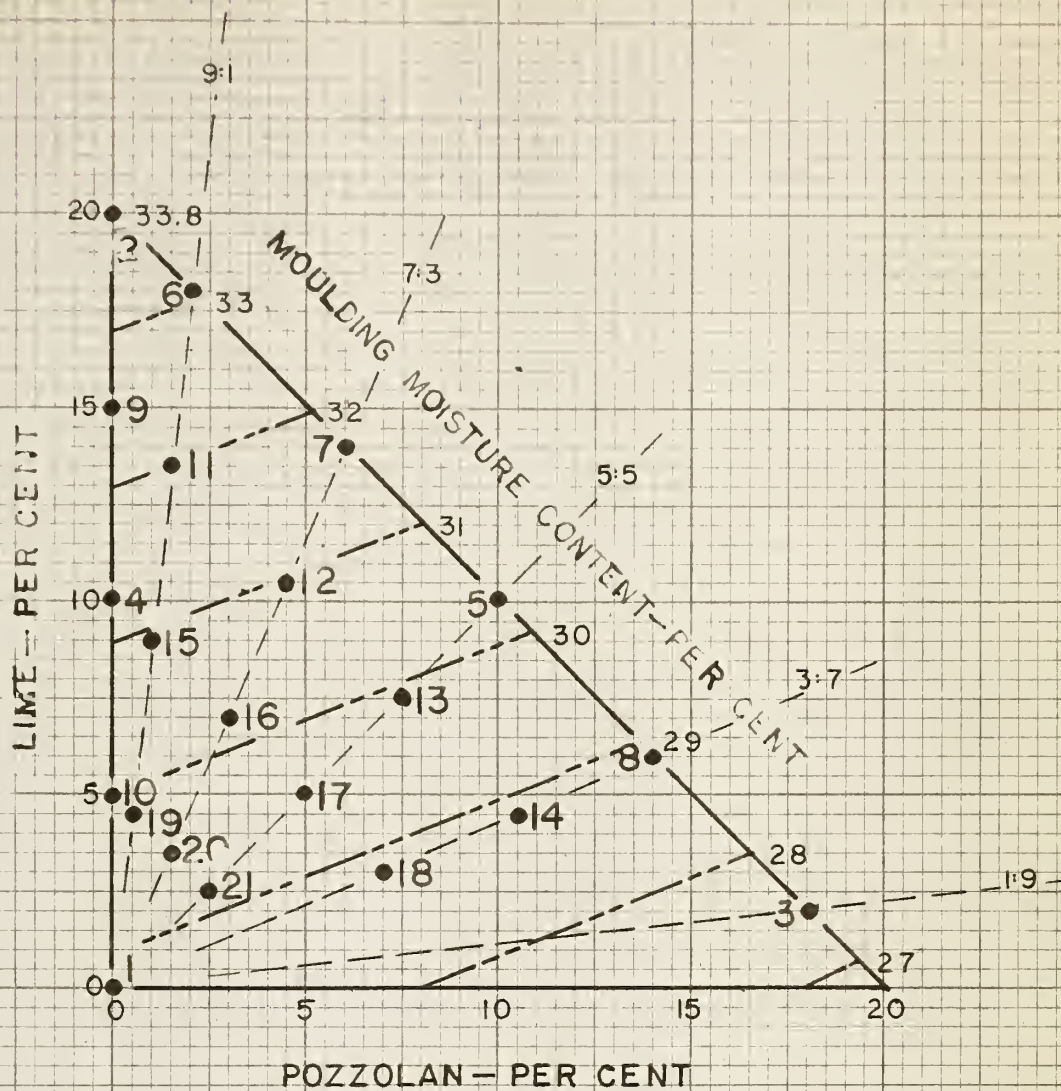


● 14 MIXTURES UNDER INVESTIGATION

WEIGHT OF DRY MIXTURE VS LIME AND POZZOLAN CONTENT  
5 BLOWS EACH FACE, 5 LB. WEIGHT

FIGURE 1





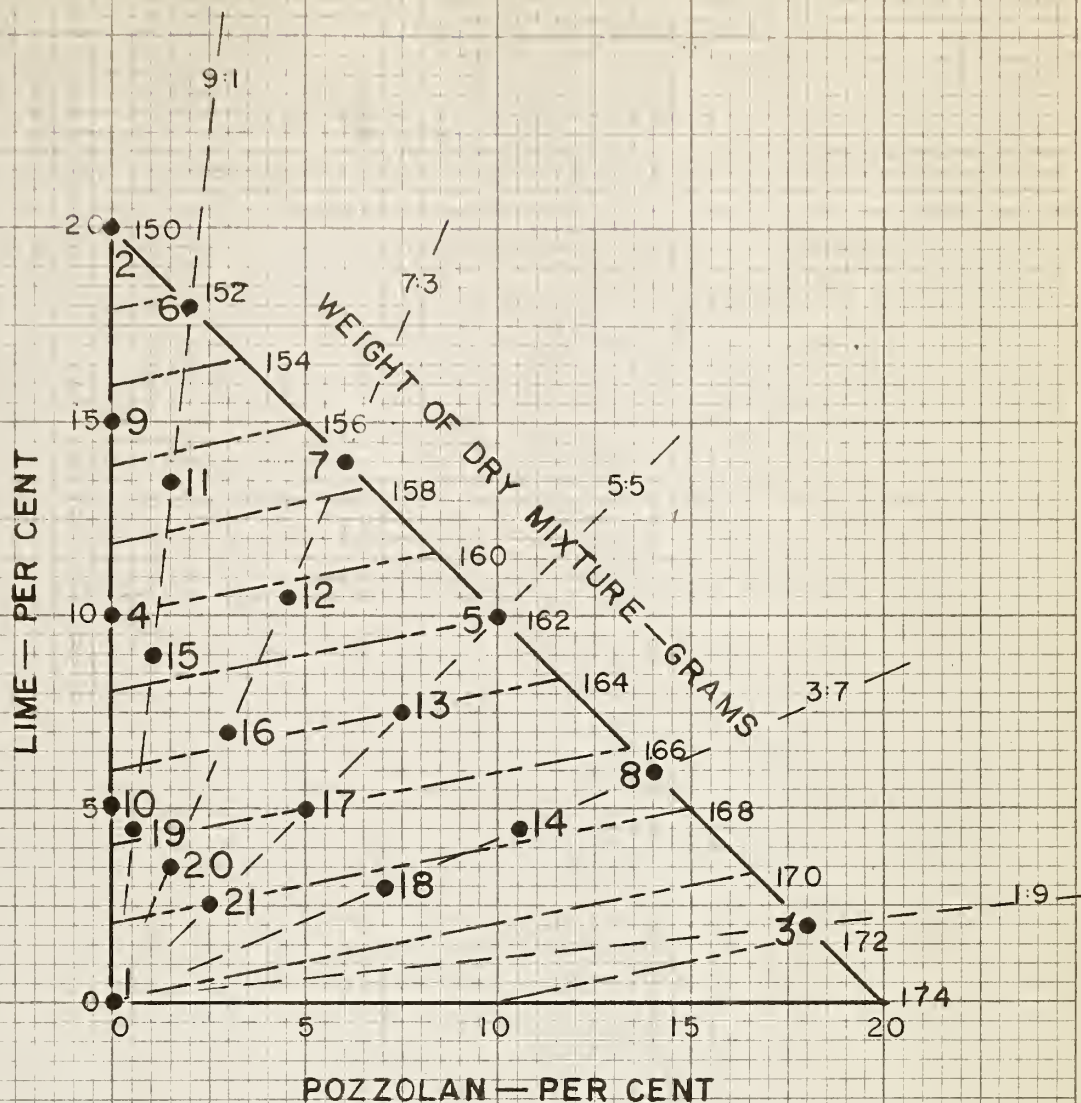
● 14 — DENOTES MIXTURES UNDER INVESTIGATION

MOULDING MOISTURE CONTENT VS LIME AND POZZOLAN CONTENT  
5 BLOWS EACH FACE, 5 L.B. WEIGHT

FIGURE 2







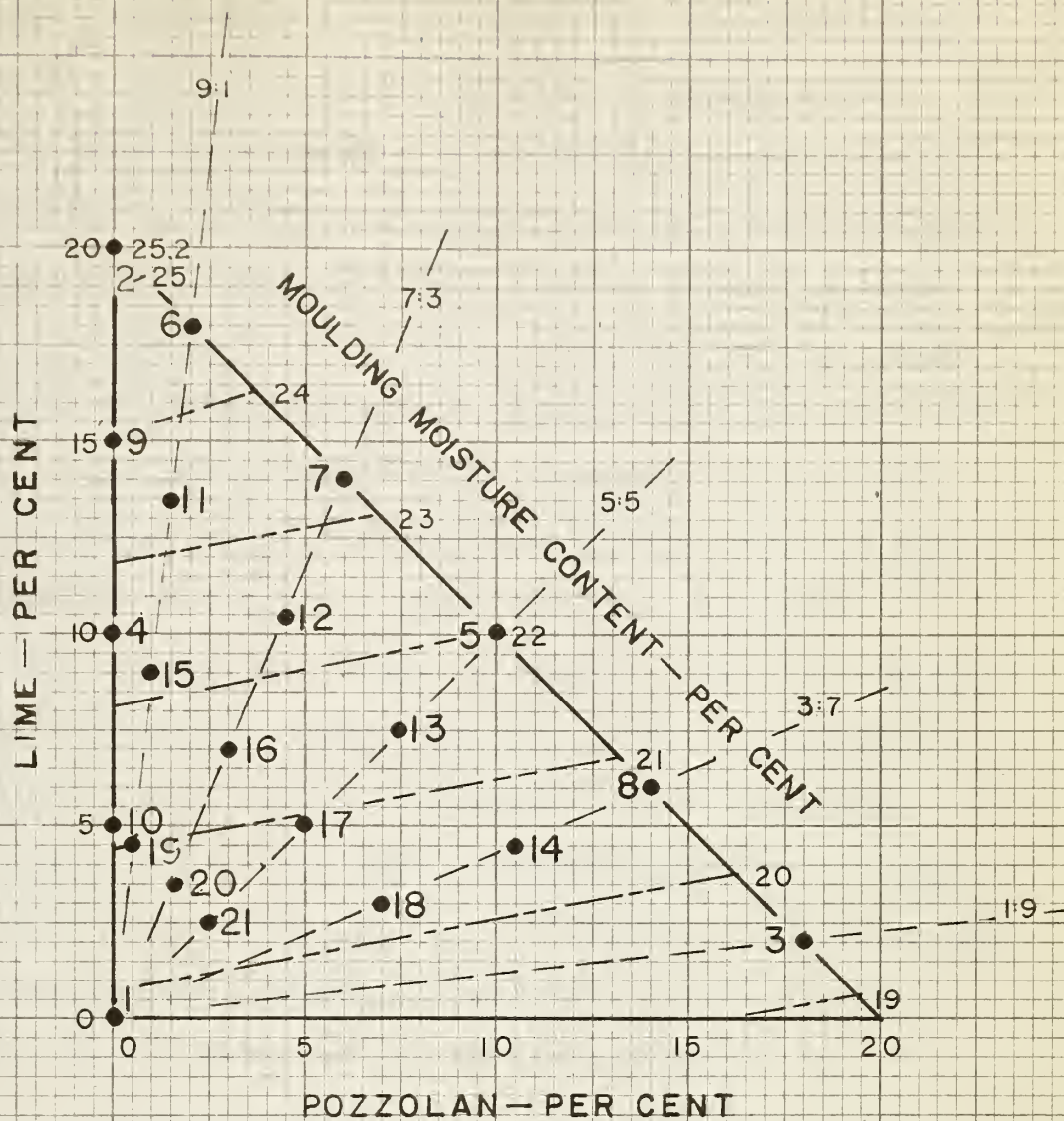
•14- DENOTES MIXTURES UNDER INVESTIGATION

WEIGHT OF DRY MIXTURE VS LIME AND POZZOLAN CONTENT  
10 BLOWS EACH FACE, 10 LB. WEIGHT

FIGURE 3







●14— DENOTES MIXTURES UNDER INVESTIGATION

MOULDING MOISTURE CONTENT VS LIME AND POZZOLAN CONTENT  
10 BLOWS EACH FACE, 10 LB. WEIGHT

FIGURE 4



indicated the freeze-thaw samples, while the specimens which were soaked in water for comparison with the freeze-thaw samples were numbered 31 to 36. For example, a sample marked 16A25, indicated the specimen contained ten per cent additive, which was the sum of seven per cent lime and three per cent pozzolan; was compacted to the lesser density; and was broken for a strength determination after 28 days of curing and 1 day of soaking. A specimen, marked 5B42 contained 20 per cent additive, comprised of equal quantities of lime and pozzolan; was compacted to the higher density; and was broken for strength after undergoing 28 days of curing, 1 day of soaking, 6 freeze-thaw cycles and 1 more day of soaking. A specimen, marked 4B31, contained 10 per cent lime, with no pozzolan added; was compacted to the higher density; and after 28 days of curing and 8 days of soaking, it was broken to determine the unconfined compressive strength. Specimens of the lesser density were given 6 daily cycles of freeze-thaw, 1 day of soaking,

The mixes marked 20 and 21 on Figure 1 and 3 were dropped from the program, because it was thought the probable strengths would have been insignificant. The results from tests 1 to 5, which were obtained before tests 20 and 21 were due to be mixed, had indicated lower strengths than were anticipated from the program. The time allotted to mix tests 20 and 21 was spent in repeating trials 2 and 4.





The program was organized so that each result was the average of six specimens. The results obtained from the program included the dry density after compaction, the change in volume caused by 24 hours of soaking, the moisture content at failure, and the unconfined compressive strength after 28 days of curing and 24 hours of soaking the specimens. In the cases of specimens compacted at the lesser density, the results also included the unconfined compressive strength and moisture content after 12 cycles of freezing and thawing; and the unconfined compressive strength, moisture content at failure and the volume change experienced by the samples after 15 days of soaking. In the cases of specimens compacted at the higher density, the results included the unconfined compressive strength and moisture content after 6 cycles of freezing and thawing; and the unconfined compressive strength, moisture content at failure, and the change in volume experienced by the samples after 8 days of soaking.

#### Accelerators For The Improvement of Strength And Durability

The accelerators which were employed in this program and the reasons for their use have already been discussed in chapter III. None of the accelerators were tested extensively in this study as only two of the 19 mixes were treated with chemical additives. These mixtures were number 2, which contained 20 per cent lime, and number 7, which contained 14 per cent lime and 6 per cent pozzolan. All of the specimens were compacted to the lesser density and the moulding moisture content and weights of dry materials were the same as for the untreated



specimens. Moisture-density tests were not performed on the mixtures treated with accelerators to check the optimum moulding moisture content. Therefore, the assumption that the dry density and optimum moisture content would remain constant after the addition of the trace chemicals was not founded upon fact.

Sodium hydroxide, potassium permanganate, sodium carbonate and calcium lignosulphonate were introduced, in the amount of  $\frac{1}{2}$  per cent by weight of the dry mixtures, into mixes 2A and 7A. The respective mixtures were marked 2C and 7C; 2D and 7D; 2E and 7E; and 2F and 7F. In addition, mixes containing two per cent calcium lignosulphonate were produced and marked 2G and 7G. The weight of the moulding water was increased for the latter mixes to compensate for the increased dry weight of the mix. All of the accelerators were dry mixed in powder or crystal form into the mixture before the mixing water was added.

The specimens were mixed, allowed to rot, were cured, and then tested according to the procedures for strength and durability outlined for the untreated specimens. 21 specimens were compacted from each mix; and the poorest of the lot, designated 10, 11 and 12, were tested for strength after 7 days of curing and 1 day of soaking. Thus the results from these tests probably indicated minimum, and not average, 7-day strengths. The specimens containing calcium lignosulphonate, in particular, were difficult to compact because of the stickiness of the material.



### IV.3 TEST PROCEDURES

#### Calculation of Mixtures

All specimens were compacted at an optimum moisture content determined from specimens which had undergone a rotting time of 24 hours between mixing and compaction. In appendix B, the procedures are given for the calculation of the weights of lime, pozzolan, air-dried soil, and distilled water required for a specimen that was 2 inches in height.

Figure 1 illustrates the weight of dry mixture required to give 2-inch specimens when compacted at the lesser density.<sup>1</sup> Figure 2 portrays the moisture content required to give this optimum condition.<sup>2</sup> Figures 3 and 4 show the same quantities for specimens compacted to the higher density.<sup>3</sup> The figures were constructed by determining the dry weights and the water contents of the three corners by experimental means; and then by calculating the values of the intermediate points by interpolation. The assumption that the functions were linear was proposed by Davidson and Handy (1960).

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<sup>1</sup>The moisture content in the air dry soil was not included in the weight. The lesser density was that obtained by dropping a 5-pound hammer through a distance of 12 inches, 5 times on each end of the specimen.

<sup>2</sup>The moisture content includes the weight of water in the air-dry soil.

<sup>3</sup>The higher density was that obtained by dropping a 10-pound hammer, through a distance of 12 inches, 10 times on each end of the specimen.





## Mixing

The weights of the lime, pozzolan and air-dried soil were estimated from Figures 1 and 3 and the weight of water added to the soil was calculated from Figures 2 and 4. Allowances were made for the moisture content of the air-dried soil, but the lime and pozzolan were assumed to be oven-dry. Sufficient quantities to produce 19 or 20 specimens were weighed to the nearest gram and the ingredients were mixed dry in a large aluminum mixing bowl. The materials were stirred by hand until blended to one color. Usually the dry mixing was completed in less than 4 minutes.

The distilled water, having been weighed to the nearest gram, was added to the dry mixture from half-liter plastic bottles, which were fitted with stoppers manufactured for the sprinkling of clothes. This phase of the mixing usually lasted 15 minutes; the mixture being stirred continuously while the water was being added. The remainder of the 30 minute period was spent in pulverizing the larger clods. At the end of the mixing time, the soil mixture was apportioned by weight to the nearest 0.1 grams; and placed into 2-pound polyethelene bags, which were then sealed.<sup>1</sup>

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<sup>1</sup>The bags were sealed in the following manner. The open end was folded down about 1 inch, the bag was placed between two 1-inch square steel bars and the fold was allowed to protrude one quarter inch. A gas flame was then applied to the fold until the fold formed a fused bead along the surface of the bars.



To obtain 18 specimens, the batch was calculated to yield enough soil for an additional specimen. The extra portion was used to determine the moisture content and the plastic limit at the end of the 24 hour rotting period.

### Compaction

The specimens were compacted between 23 and 25 hours after the time of mixing. The six specimens for the strength test were compacted in mould number 2, the six freeze-thaw control specimens in number 3, and the freeze-thaw specimens in number 4, in accordance with the procedure described in appendix B. The extrusion of the specimens was accomplished with the hydraulic apparatus shown in Plate 1a.

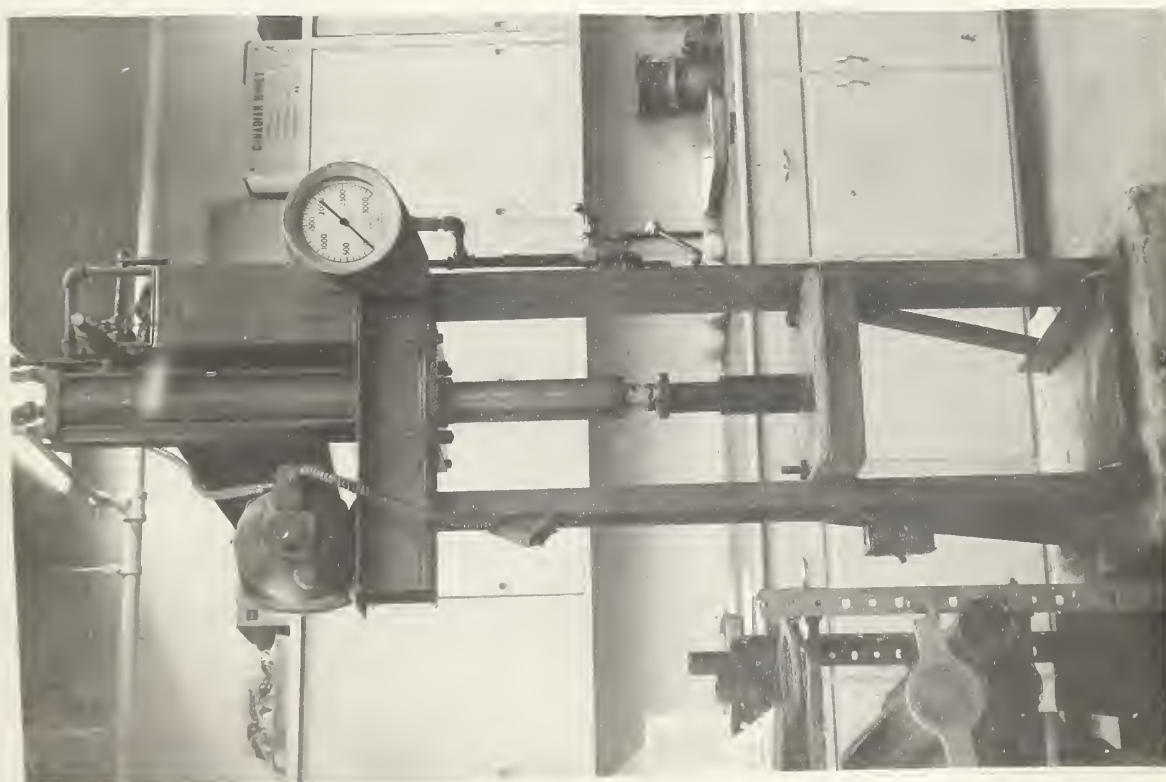
### Initial Measurements

When all of the specimens from each batch were compacted and extruded, they were measured for height and described in appendix B, and weighed to the nearest 0.10 grams. The specimens were then immediately resealed in the same bags.

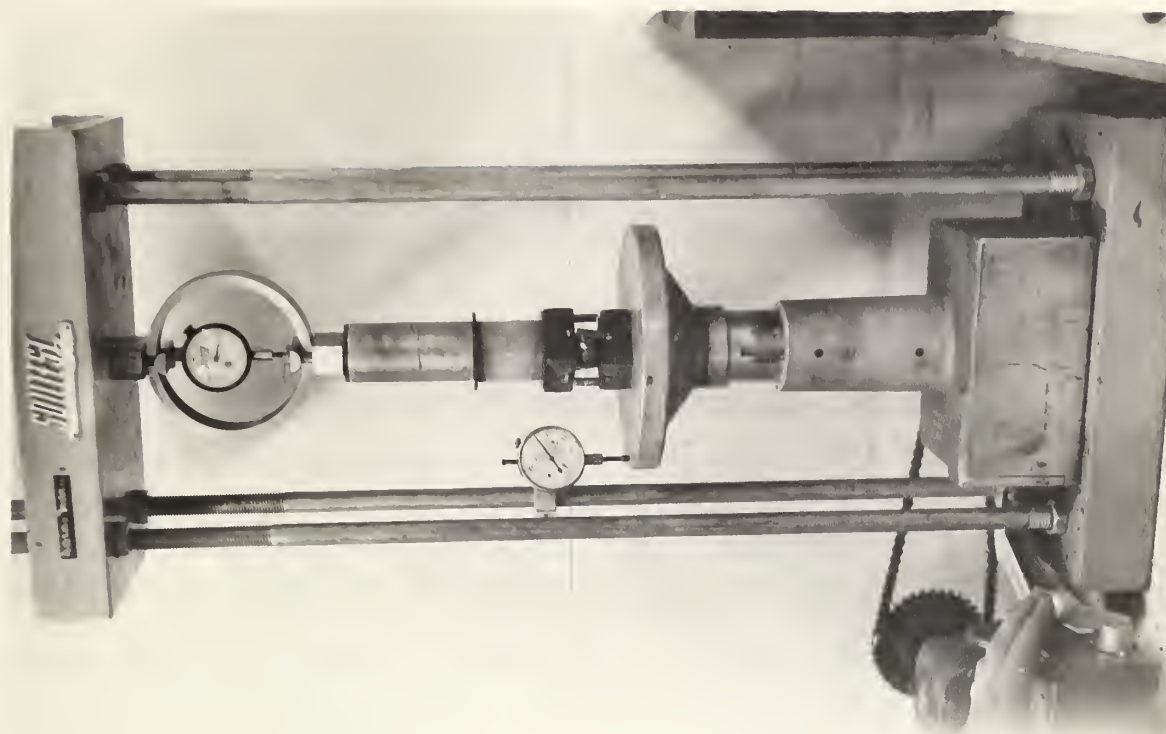
### Curing

The curing period was 28 days, plus or minus 4 hours. The specimens were stored in a moist room under temperatures which varied from 20.5C to 26C. The humidity of the room was not significant since the specimens were sealed in polyethelene bags.





EXTRUSION APPARATUS  
PLATE 1a



COMPRESSIVE STRENGTH TEST  
PLATE 1b





### Soaking

The specimens were removed from the bags at the end of the curing period. From this point in the procedure, the strength test specimens were treated in a different manner than the freeze-thaw specimens. The freeze-thaw procedure is continued under the subsection entitled Durability.

The strength specimens were immersed in distilled water for a period of 24 hours, plus or minus 2 hours. The specimens were placed in clean cake tins, which had been coated on the inside surface with two coats of Plastiglo.<sup>1</sup> The water was added until it just covered the top surface of the soil.

### Final Measurements

The diameter and height of the specimens were measured and the weight was recorded to the nearest 0.10 grams; the surface water being removed by means of a paper towel before the final measurements were taken.

The diameter was measured using the same Ames dial that was used in the initial and final measurements of height. Two diameters at right angles to each other were measured at the mid-height of the specimens and the average value was recorded to the nearest 0.001 inches. The height was measured and recorded as described in appendix B.

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<sup>1</sup>Plastiglo was a brand name for a patented acid and alkali resistant paint.



### Unconfined Compression Test

A brass plate, 0.1 inches in thickness, was placed on each end of the cylinder as it was centered under the loading head of the testing machine. The specimen was then broken at a rate of strain of 0.087 inches per minute. The arrangement of the apparatus and specimen is portrayed in Plate 1b.

The unit strain was calculated, and used in the computation of the area at failure as shown by the following equation:

$$A_c = \frac{A_o}{1 + \epsilon}$$

Where  $A_c$  was the corrected area,

$A_o$  was the area calculated from the final measurements,

and  $\epsilon$  was the unit strain at failure.

The load on the cylinder was calculated by noting the number of divisions covered by the stress dial, and multiplying this integer by the number of pounds per division picked from the calibration curve of the proving ring (appendix C).

The moisture content of the mixture was determined after failure of the specimen.

### Durability

The modified British freeze-thaw test has already been described briefly in the last section of chapter II.<sup>1</sup> Whereas

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<sup>1</sup>For details of this test, see Davidson and Bruns (1960).



the modified British test used a thermos bottle filled with water to provide a constant supply of water to the freezing specimen, the variation used in this test provided a common reservoir for 18 specimens. A wooden box that was substituted for the thermos bottle offered sufficient insulation to cause a temperature differential between the top and the bottom of the specimen. The cold source was a deep freeze with an air temperature of  $-15^{\circ}\text{C}$ , while thawing was accomplished at room temperature..

The specimens were subject to frost action, or ice segregation, because of the open system supplied by the large reservoir. The apparatus is described in appendix D. The detailed procedures are given in the following paragraphs.

Soaking of the specimens. The specimens were wrapped in a polyethelene liner, which extended  $\frac{1}{4}$  inch above the top face and was flush to the bottom face. The bottom plates were fastened to the moulds and filter papers were added to prevent any soil from being lost through the holes in the plates. The tray, full of specimens, was placed in a large steel pan, in which the water level was maintained slightly below the top of the specimens. The soaking period varied between 12 and 22 hours.

The freeze-thaw cycles. At the end of the initial soaking period the lids containing the specimens were placed on







SPECIMENS IN FREEZER

PLATE 2



the boxes and the tops of the specimens were painted with a thin coat of type RC-0 asphalt. The level of the distilled water in the boxes was maintained  $\frac{1}{4}$  inch above the bottom of the specimens. The boxes were kept in the deep freeze for periods which varied from  $12\frac{1}{2}$  to  $13\frac{1}{2}$  hours. The thaw cycle was approximately 11 hours in duration.

The samples of the higher density were removed after 6 cycles of freezing and thawing, while those of the lesser density were removed after 12 cycles. Thus at the end of every 6 cycle period, six B specimens and six A specimens of different batches would be replaced by six A and six B specimens from a common mix. The six A specimens that were not changed in each tray were given a soaking period between the sixth and seventh cycles. Thus all of the A specimens were subjected to a period of soaking, six cycles of freeze-thaw, a period of soaking, and six more cycles of freeze-thaw. The program was arranged in this manner to accommodate the equipment available. The boxes were filled with fresh water for each change in specimens.

The procedure made it impossible to compare the effect of compacted dry density upon the durability of the mixtures.

Extrusion of the specimens from the moulds. After the completion of the final thawing cycle of the specimens, the samples were extruded and immersed in water. The extrusion was accomplished by removing the bottom plate in order to push the bottom face. When the sample was held too tightly by the mould



to be extruded by hand, a testing machine had to be used to force the specimen from the mould.

Soaking and determination of strength. The specimens were soaked, without the polyethelene wrappers, in distilled water for 24 hours. The diameter of the specimens was measured again and the samples were tested for strength in accordance with the procedures given for the 28-day strength specimens. In the case of the durability tests the strength was based on the area before the application of load and not on an area corrected for strain.

Control tests. The freeze-thaw control specimens were immersed in water the same day that their counterparts were placed in the trays, and remained there until they were broken the same day as the companion specimen which had undergone the durability test.

The specimens were measured for height and for diameter and were weighed in a surface dry condition before being placed in the testing apparatus. The strength was based on the area of the specimen before the application of the load.

Index of Resistance to the Effect of Freezing. The Index of Resistance to the Effect of Freezing ( $R_f$ ) was calculated by the formula:

$$R_f = \frac{100 P_f}{P_c} \quad (\%)$$





Where  $P_f$  was the failure stress of the freeze-thaw specimen,

and  $P_c$  was the failure stress of the control specimen.

#### IV.4 DISCUSSION OF PROCEDURES

##### Time of Mixing

Attempts to mix the clay soil and water in a mechanical mixer were unsuccessful because the clay tended to form large clods. The mixing time of 4 minutes used by Hvozdanski was too short when the sample was mixed by hand. To ensure a thorough mix for every soil-lime mixture the time taken to thoroughly hand-mix the raw soil for the specimens marked 1A was taken as the standard. Many of the mixtures with high lime contents were well mixed in 20 minutes, but the mixing was continued for the full half hour for all of the batches.

##### Compactive Effort

Hvozdanski achieved higher densities, near those obtained from a modified AASHO moisture-density test, by increasing the number of blows, with the 5 pound hammer on each end of the specimen, from 5 to 25 and 50 blows. Viskochil and others (1957) found that a 10 pound weight, dropped 10 times on each end of sand, clay or silt specimens, gave the same dry density as the modified AASHO test. Tests run on the Fahler clay using a 10 pound weight, dropped 10 times on each end of the specimens, gave approximately 95 per cent of the



modified AASHO value. The details of the results are portrayed in appendix B.

Some trials were conducted in which the number of blows with the 10 pound hammer were increased. The results indicated that the compactive effort would have to be doubled in order to reach the modified density. The difference between the standard proctor density and 95 per cent of the modified density was sufficient to show the relationship between strength and compactive effort, and the extra effort which would have been required to reach the modified density was not needed for this investigation.

#### The Rotting Period

All of the specimens experienced a rotting period of approximately 24 hours between the times of mixing and compacting. The results of varying the rotting time from zero to 24 hours are shown in appendix B. The results showed that a rotting period reduced the dry density of the specimens of mixtures containing clay and 20 per cent lime when they were compacted to the lesser density. The results of the same mixture, when compacted at the higher density, indicated that the rotting time had no effect on the dry density. Mitchell and Hooper (1960) and Davidson and Ruff (1960) reported decreased densities and decreased strengths in specific lime-clay mixtures that were attributed to delayed compaction.

Since clays and lime are difficult to mix, delays in



the field compaction of a clay-lime mixture may be the general, rather than the exceptional, procedure. Therefore the extra strength available from immediate compaction may not be present in all field installations. The waiting period of 24 hours also allowed more flexibility in the testing program. For instance, a difference of 1 hour in the 24 hour rotting period only indicated a deviation of 8 per cent from the specified rotting period.

The rotting period also enabled the soil and lime to aggregate before compaction, thus reducing the plasticity of the soil. The benefits from the rotting time would likely have been increased if the mixture had been stirred for a moment in the mould, just prior to compaction.

#### Soaking of the Specimen

The samples were broken after they had soaked for a period of about 24 hours. The only strength left to resist deformation came from the cementing action of the lime, since the saturated specimen had no natural strength. To prove this statement, three extra specimens were moulded for all of the mixes from 1A to 7A inclusive, and 1B to 7B inclusive. All of the specimens, which were immersed in water immediately after compaction, disintegrated within 24 hours.

#### Calculation of the Change in Volume

The change in volume was calculated to discover the relationship between the swelling characteristics and the strength of the soil.





## The Durability Test

The preparation of the specimens. The modified British test procedure was followed unless it proved to be unworkable or inconvenient for the present investigation. Two-mil polyethelene was used as a liner in the moulds instead of the Saran wrap mentioned in appendix A of Davidson and Bruns (1960). It offered less resistance to vertical movement of the soil in the mould than when the Saran wrap was used or when no wrapping at all was used.

It is suspected that one of the reasons for specifying the Saran wrap was the ability of the material to seal the specimen from loss of moisture and from entry of carbon dioxide. In this respect, the polyethelene was inferior to the Saran wrap. Unfortunately, this suspicion was not aroused until a major part of the program was completed; and another change in the program at that time was not feasible.

The tops of the specimens were treated with asphalt to prevent drying of the surface during the thaw periods. It was suspected that the asphalt also provided a seal against entry of carbon dioxide. The asphalt was not applied until the completion of the first soaking period because it might have entrapped the air in the top of the specimen, and prevented complete saturation.

The specimens were soaked after they were placed in the moulds, because many of the specimens would have swollen to such



a large diameter that they could not have been placed into the moulds.

Freezing and thawing. The length and severity of the freezing cycles were established by running pilot tests in a small freezer chest with a capacity of two boxes. Thermocouples were attached to the bottom and top faces of two specimens treated with 10 per cent lime. A 25-watt aquarium heater was operated continuously in the reservoir; and the water temperature in the reservoir and the air temperature in the chest were read simultaneously with the temperatures of the thermocouples. The frost line had penetrated to the bottom of the specimens between 12 and 13 hours at an air temperature of  $-20^{\circ}\text{C}$ .

The testing program proper was carried out in a commercial deep freeze, which could easily accommodate five freeze-thaw boxes. Unfortunately, the testing program began before pilot tests could be run in the deep freeze. As a result, some of the early tests, particularly tests 5 to 11, were subjected to quite irregular freezing cycles while the thermostat, which controlled the air temperature in the freezer, was being calibrated. The minimum air temperature that could be maintained in the deep freeze was  $-15^{\circ}\text{C}$ , because the small refrigeration unit could not cope with the large heat capacity of the five boxes in the short time of 13 hours. The lids did not fit tightly on the boxes and heat from the reservoirs



escaped into the freezer compartment. On August 2, 1961, the heaters were removed from the boxes, as experiments in which the heaters were disconnected at various times throughout the cycle, showed that the heaters were unnecessary. Thus only test 17 and subsequent tests received a standard test in which one could be assured of penetration of the frost line through the full length of the specimen. Tests 2 and 4 received this standard test because they were repeated near the completion of the program.

The heaters were originally installed to prevent ice from forming in the corners and edges of the boxes during the freezing cycle. In the commercial deep freeze this problem did not arise.

Removal and testing of the soil specimens. The diameter and degree of roughness of the moulds varied considerably. In addition, some specimens tended to swell more than others. Thus the inside surface of the moulds offered a variable resistance to frost heaving and an even greater resistance to settlement during the thaw periods. The resistance of the moulds was probably the largest variable in determining the durability of the mixtures. This variable could have been eliminated by increasing the size of the moulds.

The asphalt coating and polyethelene wrapping usually held the specimens intact during their extrusion. However, in unwrapping the specimens, the asphalt which adhered to the





polyethelene would sometimes remove part of the soil sample, if the asphalt coating was too thick. The asphalt topping also tended to float away when the specimens were soaked in water after their extrusion from the moulds. The thickness of the asphalt coating was reduced for all of the specimens in test 16 and subsequent tests. Reducing the thickness of the asphalt coat stopped the loss of soil during the unwrapping operation, but it did not prevent the loss of the tops during the soaking process. The loss of the asphalt top was invariably followed by softening and partial disintegration of the top of the specimens. These specimens could not be properly tested for strength because of the uneven nature of the top surface. The loss of the asphalt tops and subsequent disintegration of the specimens reduced the number of specimens fit to test and in some cases all of the specimens were lost in this manner.

Freezing and thawing caused ice segration in the specimens. When the axial load was applied, an unknown amount of the vertical movement consisted of closing the horizontal cracks which extended through the specimens. Since this lineal strain was not accompanied by a corresponding expansion in the area, the area was not corrected before computation of the failure stress.

The final height of the freeze-thaw specimens had little significance because it included an unknown thickness of asphalt.



## IV.5 ESTIMATE OF ERROR IN RESULTS

The quality of the results can be assessed by the calculation of the mean for six specimens, and by establishing a degree of confidence in each answer.<sup>1</sup> The following tabular presentation represents the average values of specimens 5B20 through 5B26. One can be 95 per cent confident that the true mean of an unlimited number of specimens would be within the limits shown.

<u>Calculation</u>	<u>Result</u>	<u>Units</u>
Wet density as compacted	115.7 $\pm$ 0.57	pounds per cubic foot
Dry density as compacted	93.5 <sup>2</sup>	pounds per cubic foot
Volume change	3.05 $\pm$ 0.48	per cent
Dry density at failure	91.44 $\pm$ 0.48	pounds per cubic foot
Unconfined compressive strength	280 $\pm$ 23	pounds per square inch
Moisture content at failure	28.7 $\pm$ 0.5	per cent

---

<sup>1</sup>The confidence interval is calculated from the equation:

$$L_1 = \bar{Y} - t_{.05} S_Y$$

where  $L_1$  is the lower limit of confidence

$\bar{Y}$  is the mean of the sample

$S_Y$  is the standard deviation

and  $t_{.05}$  is a value extracted from appendix 6 of Ostle (1954)

For 21 specimens  $t = 2.086$

for 18 specimens  $t = 2.110$

for 6 specimens  $t = 2.571$

for 3 specimens  $t = 4.303$

In each case  $t$  can be applied to the 95% confident level only.

<sup>2</sup>The accuracy of the single moisture content was unknown and therefore the accuracy of the dry density was also an unknown quantity.



### Precision Of Calculations

All of the calculations were completed on a slide rule with the exception of the volumetric calculations. To retain four significant figures through most of the computations, the volumes were computed by means of a desk calculator.





## CHAPTER V

### RESULTS OF THE TEST PROGRAM

The significant results, accumulated in the test program, are presented in Tables I to VII. Tables I and II display the average values of the 28-day strengths for the stabilized clay-lime-pozzolan specimens which had been soaked the full day before the strength tests. Table III shows the average unconfined compressive strengths of the freeze-thaw control specimens which had soaked for fifteen days, while Table IV exhibits the strengths of the control specimens which were immersed in water for eight days. The durability of the freeze-thaw samples is reported in Tables V and VI. The effect of accelerators on the strength of two of the mixtures is indicated in Table VII.

The Tables also report the volume changes experienced during the soaking periods, the moisture contents at failure, the dry densities as compacted, the moulding moisture content, and the voids-cement ratios; the cement in this investigation being the lime-pozzolan additive to the clay soil.

A discussion of the effect of the lime and pozzolan upon the unconfined compressive strength is presented in which each variable is considered independently.

A critical study of the durability tests concludes the chapter.

The original data, because of its bulkiness is not



## SUMMARY OF RESULTS

TABLE I

## 28-DAY STRENGTH TESTS OF SPECIMENS COMPACTED

WITH A 5-LB. WEIGHT DROPPED 5 TIMES ON EACH FACE

Lime-Pozzolan Ratio	Additive Content %	Mix No.	Stress psi	Initial Dry Density pcf	Initial Moisture Content %	Final Moisture Content %	Increase in Volume %	Voids-Additive Ratio
10:0	0	1A	---	87.6	32.7	---	---	---
9:1	20	2A	142	81.6	34.1	37.5	3.68	6.67
7:3	20	6A	142	79.5	37.3	32.0	2.20	7.02
5:5	20	7A	173	81.9	31.0	37.2	3.16	6.64
3:7	20	5A	267	83.9	32.4	32.7	2.01	6.34
1:9	20	8A	176	84.6	27.7	34.6	1.53	6.24
10:0	15	3A	23	86.1	27.8	45.0	2.44	6.02
9:1	15	9A	142	82.5	32.1	37.1	3.03	8.33
7:3	15	11A	174	84.4	29.8	35.3	3.51	8.01
5:5	15	12A	159	83.9	29.9	36.2	3.08	8.10
3:7	15	13A	141	83.6	29.5	35.5	2.98	8.12
10:0	10	14A	148	84.5	29.1	34.1	0.97	7.98
9:1	10	4A	136	83.6	30.5	36.0	6.78	11.69
7:3	10	15A	128	84.2	30.2	35.2	4.38	11.53
5:5	10	16A	126	84.7	30.4	33.9	5.62	11.39
3:7	10	17A	126	85.0	28.8	33.8	4.73	11.31
10:0	5	18A	89	85.5	28.8	33.7	0.48	11.17
9:1	5	10A	84	85.4	28.5	34.7	3.61	21.45
		19A	108	84.8	28.9	34.7	2.18	21.72

1 - By weight

2 - Percentage of weight of dry soil

3 - Pounds per square inch

4 - Pounds per cubic foot

5 - By volume



TABLE II

## 28-DAY STRENGTH TESTS OF SPECIMENS COMPACTED

WITH A 10-LB. WEIGHT DROPPED 10 TIMES ON EACH FACE

Lime-Pozzolan Ratio	Additive Content %	Mix No.	Stress psi	Initial Dry Density pcf	Initial Moisture Content %	Final Moisture Content %	Increase In Volume %	Voids-Additive Ratio
10:0	0	1B	---	100.8	21.8	---	---	---
9:1	20	2B	210	90.5	25.7	31.5	4.44	5.44
7:3	20	6B	265	91.5	25.3	27.5	3.56	5.30
5:5	20	7B	238	92.6	22.6	30.3	4.07	5.16
3:7	20	5B	280	93.5	23.6	28.7	3.05	5.06
1:9	20	8B	171	95.3	19.5	29.2	4.53	4.86
10:0	15	3B	---	97.3	20.8	---	---	---
9:1	15	9B	192	93.0	23.0	30.0	4.22	5.56
7:3	15	11B	185	92.5	22.2	30.1	4.34	6.62
5:5	15	12B	168	93.9	21.6	29.7	4.62	6.42
3:7	15	13B	154	93.7	20.8	30.4	5.08	6.45
10:0	10	14B	147	95.3	20.0	28.7	4.18	6.21
9:1	10	4B	173	94.8	22.2	29.0	8.03	9.05
7:3	10	15B	134	93.9	21.4	29.5	7.23	9.25
5:5	10	16B	105	95.1	21.3	29.4	6.15	9.27
3:7	10	17B	103	96.2	19.8	28.7	8.89	9.72
10:0	5	18B	89	96.8	19.8	28.0	2.91	8.61
9:1	5	10B	60	95.0	20.3	30.9	8.11	17.19
		19B	80	96.1	20.3	30.2	7.35	16.77

- 1 - By weight
- 2 - Percentage of weight of dry soil
- 3 - Pounds per square inch
- 4 - Pounds per cubic foot
- 5 - By volume





## SUMMARY OF RESULTS

TABLE III

## TESTS OF FREEZE-THAW CONTROL SPECIMENS COMPACTED

WITH A 5-LB. WEIGHT DROPPED 5 TIMES ON EACH FACE

Lime- Pozzolan Ratio <sup>1</sup>	Additive Content % <sup>2</sup>	Mix No.	Stress psi <sup>3</sup>	Initial Dry Density pcf <sup>4</sup>	Initial Moisture Content %	Final Moisture Content %	Increase In Volume %	Voids- Additive Ratio <sup>5</sup>
10:0	0	1A	---	87.6	32.7	---	---	---
9:1	20	2A	160	81.6	34.1	40.8	5.62	5.67
7:3	20	6A	132	79.5	37.3	38.8	5.23	7.02
5:5	20	7A	179	81.9	31.0	39.0	7.58	6.64
3:7	20	5A	225	83.9	32.4	35.8	3.90	6.34
1:9	20	8A	207	84.6	27.7	37.1	2.53	6.24
10:0	15	3A	---	86.1	27.8	---	---	6.02
9:1	15	9A	154	82.5	32.1	40.4	5.90	8.33
7:3	15	11A	191	84.4	29.8	39.6	5.70	8.01
5:5	15	12A	172	83.9	29.9	38.4	6.52	8.10
3:7	15	13A	175	83.6	29.5	38.4	4.89	8.12
10:0	10	14A	151	84.5	29.1	35.1	1.54	7.98
9:1	10	4A	141	83.6	30.5	40.5	7.44	11.62
7:3	10	15A	146	84.2	30.2	38.9	5.94	11.53
5:5	10	16A	168	84.7	30.4	37.4	7.85	11.39
3:7	10	17A	114	85.0	28.8	36.5	5.67	11.31
10:0	5	18A	90	85.5	28.9	35.0	0.65	11.17
9:1	5	10A	62	85.4	28.5	38.6	6.82	21.45
	5	19A	83	84.8	28.9	36.3	3.77	21.72

1 - By weight

2 - Percentage of weight of dry soil

3 - Pounds per square inch

4 - Pounds per cubic foot

5 - By volume

14A - broken 1 day too early



SUMMARY OF RESULTS

TABLE IV

TESTS OF FREEZE-THAW CONTROL SPECIMENS COMPACTED  
WITH A 10-LB. WEIGHT DROPPED 10 TIMES ON EACH FACE

Lime- Pozzolan Ratio	Additive Content %2	Mix No.	Stress psi3	Initial Dry Density pcf4	Initial Moisture Content %	Final Moisture Content %	Increase In Volume %	Voids- Additive Ratio5
10:0	0	1B	---	100.8	21.8	---	---	---
9:1	20	2B	169	90.5	25.7	34.7	7.54	5.44
7:3	20	6B	202	91.5	25.3	32.3	5.97	5.30
5:5	20	7B	189	92.6	22.6	31.9	6.86	5.16
3:7	20	5B	246	93.5	23.6	31.2	5.75	5.06
1:9	20	8B	171	95.3	19.5	30.7	6.49	4.85
10:0	15	3B	---	97.3	20.8	---	---	---
9:1	15	9B	149	93.0	23.0	33.8	8.30	6.56
7:3	15	11B	142	92.5	22.2	33.1	10.24	6.62
5:5	15	12B	157	93.9	21.6	31.6	9.66	6.42
3:7	15	13B	158	93.7	20.8	33.4	8.49	6.45
10:0	10	14B	119	95.3	20.0	31.2	7.68	6.21
9:1	10	4B	144	94.8	22.2	32.5	10.30	9.03
7:3	10	15B	127	93.9	21.4	32.9	8.59	9.25
5:5	10	16B	109	95.1	21.3	33.7	9.80	8.97
3:7	10	17B	94	96.2	19.7	31.4	9.12	8.72
10:0	5	18B	81	96.8	19.8	28.7	4.07	8.61
9:1	5	10B	45	95.0	20.3	34.5	14.02	17.19
	5	19B	51	96.1	20.3	33.2	12.00	16.77

- 1 - By weight
- 2 - Percentage of weight of dry soil
- 3 - Pounds per square inch
- 4 - Pounds per cubic foot
- 5 - By volume



TABLE V

## TESTS OF FREEZE-THAW SPECIMENS COMPACTED WITH

A 5-LB. WEIGHT DROPPED 5 TIMES ON EACH FACE.

Lime-Pozzolan Ratio	Additive Content % <sup>2</sup>	Mix No.	Stress psi <sup>3</sup>	Initial Dry Density pcf <sup>4</sup>	Initial Moisture Content %	Final Moisture Content %	Increase In Volume %	Voids-Additive Ratio <sup>5</sup>
10:0	0	1A	---	87.6	32.7	---	---	---
9:1	20	2A	---	81.6	34.1	---	---	6.67
7:3	20	6A	28	79.5	37.3	44.0	---	7.02
5:5	20	7A	60	81.9	30.0	42.5	---	6.64
3:7	20	5A	88	83.9	32.4	40.5	---	6.34
1:9	20	8A	115	84.6	27.7	39.6	---	6.24
10:0	15	3A	---	86.1	27.8	---	---	6.02
9:1	15	9A	58	82.5	32.1	42.2	---	8.33
7:3	15	11A	57	84.4	28.9	45.2	---	8.01
5:5	15	12A	91	83.9	29.9	41.4	---	8.10
3:7	15	13A	106	83.6	29.5	40.0	---	8.12
10:0	10	14A	29	84.5	29.1	40.5	---	7.98
9:1	10	4A	---	83.6	30.5	---	---	11.69
7:3	10	15A	---	84.2	30.2	---	---	11.53
5:5	10	16A	---	84.7	30.4	---	---	11.39
3:7	10	17A	---	85.0	28.8	---	---	11.31
10:0	5	18A	10	85.5	28.8	42.6	---	11.17
9:1	5	10A	6	85.4	28.5	49.2	---	21.45
	5	19A	---	84.8	28.9	52.0	---	21.72

- 1 - By weight
- 2 - Percentage of weight of dry soil
- 3 - Pounds per square inch
- 4 - Pounds per cubic foot
- 5 - By volume





TABLE VI  
TESTS OF FREEZE-THAW SPECIMENS COMPACTED WITH  
A 10-LB. WEIGHT DROPPED 10 TIMES ON EACH FACE

Lime- Pozzolan Ratio	Additive Content %2	Mix No.	Stress psi3	Initial Dry Density pcf4	Initial Moisture Content %	Final Moisture Content %	Increase In Volume %	Voids- Additive Ratio5
10:0	20	1B	---	100.8	21.8	---	---	---
9:1	20	2B	---	90.5	25.7	---	---	5.44
7:3	20	6B	87	91.5	25.3	35.0	---	5.30
5:5	20	7B	158	92.6	22.6	34.2	---	5.16
3:7	20	5B	136	93.5	23.6	33.1	---	5.06
1:9	20	8B	164	95.3	19.5	31.3	---	4.86
10:0	15	3B	---	97.3	20.8	---	---	---
9:1	15	9B	154	93.0	23.0	33.0	---	6.56
7:3	15	11B	---	92.5	22.2	---	---	6.62
5:5	15	12B	131	93.9	21.6	32.1	---	6.42
3:7	15	13B	106	93.7	20.8	34.1	---	6.45
10:0	10	14B	51	95.3	20.0	34.7	---	6.21
9:1	10	4B	---	94.8	22.2	---	---	9.03
7:3	10	15B	14	93.9	21.4	43.3	---	9.25
5:5	10	16B	---	95.1	21.3	---	---	8.97
3:7	10	17B	---	96.2	19.7	---	---	8.72
10:0	5	18B	9	96.8	19.8	31.6	---	8.61
9:1	5	10B	24	95.0	20.3	35.0	---	17.19
		19B	14	96.1	20.3	35.5	---	16.77

- 1 - By weight
- 2 - Percentage of weight of dry soil
- 3 - Pounds per square inch
- 4 - Pounds per cubic foot
- 5 - By volume



TABLE VII

## SPECIMENS CONTAINING ACCELERATORS

Lime- Pozzolan Ratio <sup>1</sup>	Additive Content % <sup>2</sup>	Mix No.	Stress 7 Day psi <sup>3</sup>	Stress 28 Day psi <sup>3</sup>	Initial Dry Density pcf <sup>4</sup>	Initial Moisture Content %	Final Moisture Content %	Increase In Volume %	Voids- Additive Ratio <sup>5</sup>
28-day strength		2A	64	142	81.6	34.1	37.5	3.68	
		2C		133	81.0	32.4	37.9	2.93	
10:0	20	2D	55	111	79.6	32.9	40.0	3.65	
		2E	64	132	80.8	32.4	37.2	3.19	
		2F	53	109	82.8	32.0	35.8	3.63	
		2G	37	56	82.4	32.5	36.3	1.84	
Freeze-thaw Control specimens		2A		160	81.6	34.1	40.3	5.62	
		2C		175	81.0	32.4	40.2	5.41	
		2D		143	79.6	32.9	42.4	6.31	
10:0	20	2E		170	80.8	32.4	39.9	5.68	
		2F		117	82.8	32.0	40.5	6.37	
		2G		42	82.4	32.5	42.2	6.51	
28-day strength specimens		7A	64	173	81.9	31.0	37.2	3.16	
		7C		122	81.5	30.7	37.5	2.33	
7:3	20	7D	73	147	80.8	31.2	37.5	2.83	
		7E	76	161	82.5	30.6	36.0	2.96	
		7F	55	122	84.5	30.0	35.3	3.47	
		7G	39	59	84.0	30.3	35.0	1.26	
Freeze-thaw Control specimens		7A		179	81.9	31.0	39.0	7.58	
		7C		194	81.5	30.7	39.8	4.08	
7:3	20	7D		185	80.8	31.2	41.7	5.31	
		7E		231	82.5	30.6	38.8	5.13	
		7F		131	84.5	30.0	40.1	7.38	
		7G		46	84.0	30.3	40.4	6.14	

1- By weight

2- Percentage of weight of dry soil

3- Pounds per square inch

4- Pounds per cubic foot

5- By volume

2A &amp; 7A - no accelerators

2C &amp; 7C - 0.5% NaOH

2D & 7D - 0.5% KMnO<sub>4</sub>2E & 7E - 0.5% Na<sub>2</sub>CO<sub>3</sub>

2F &amp; 7F - 0.5% calcium lignosulphonate

2G &amp; 7G - 2.0% calcium lignosulphonate



included in this report, but is retained in the archives of the Research Council of Alberta, Highways Division, Edmonton, Alberta. Sample data sheets are displayed in appendix F of this study.

## V.1 DISCUSSION OF STRENGTH RESULTS

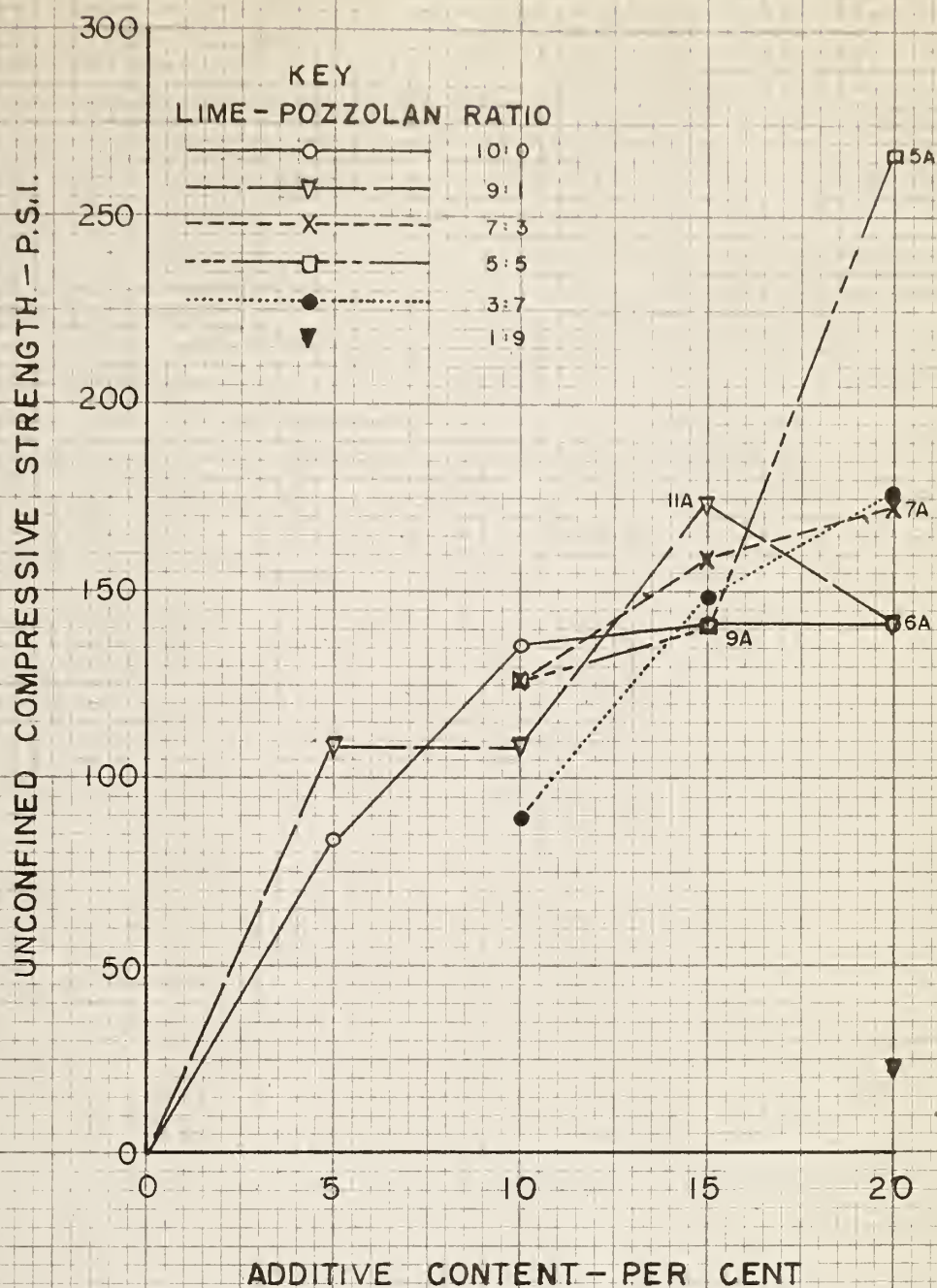
The 28-day strengths in all cases were below the value of 500 psi suggested by Davidson and Handy (1960) as the minimum strength to prevent damage from repeated cycles of freezing and thawing. The lime and pozzolan did increase the immersed strengths from zero, which was the value for untreated soil, to magnitudes of more than 250 psi for some mixtures which contained 20 per cent additive. Thus one could say that lime stabilization achieved a limited success with a clay soil. The following discussion describes how each factor affected the strength of the stabilized mixtures.

### The Content of Additive

Figures 5 and 6 reveal a substantial increase in strength with increasing additive content. Only three of the mixtures compacted to the lesser density failed to increase in strength when the additive content was increased. Mixtures containing lime only appeared to have reached a peak value of strength at lime contents between 15 and 20 per cent. When compacted to a higher dry density by increasing the compactive effort, however, the lime content which gave the highest strength using





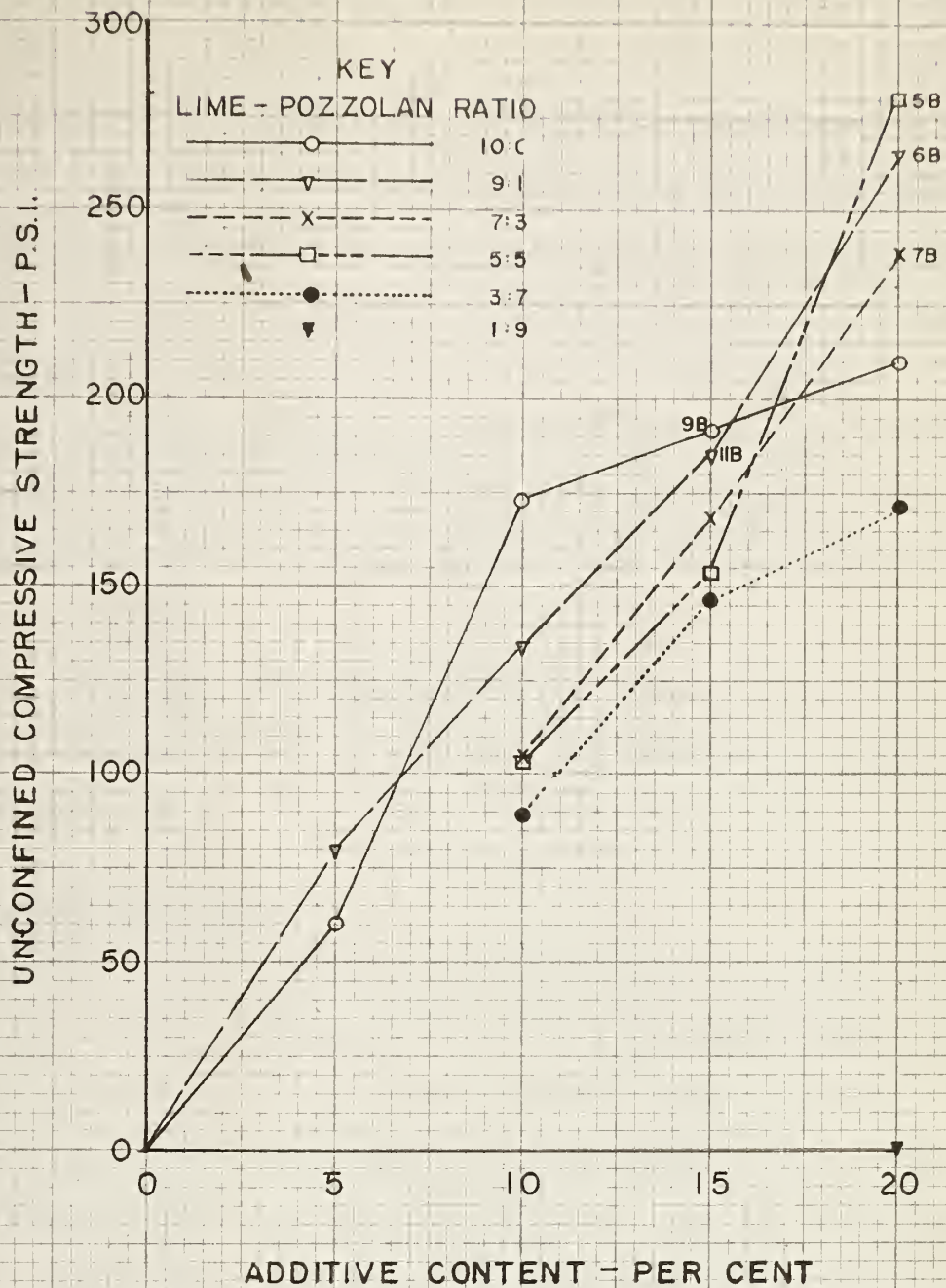


UNCONFINED COMPRESSIVE STRENGTH VS ADDITIVE CONTENT

5 BLOWS EACH FACE, 5 LB. WEIGHT  
CURING TIME 28 DAYS; SOAKING TIME-1 DAY

FIGURE 5





UNCONFINED COMPRESSIVE STRENGTH VS ADDITIVE CONTENT

10 BLOWS EACH FACE, 10 LB. WEIGHT  
CURING TIME, 28 DAYS; SOAKING TIME 1 DAY

FIGURE 6





the same mixtures seemed to shift to some value greater than 20 per cent.

Because the results were contrary to the general trends indicated in Figures 5 and 6, one can assume that the strengths of specimens from mixture 11A were too high or else the strengths of 9A were too low, due to some factors which were not evident in the Figures.

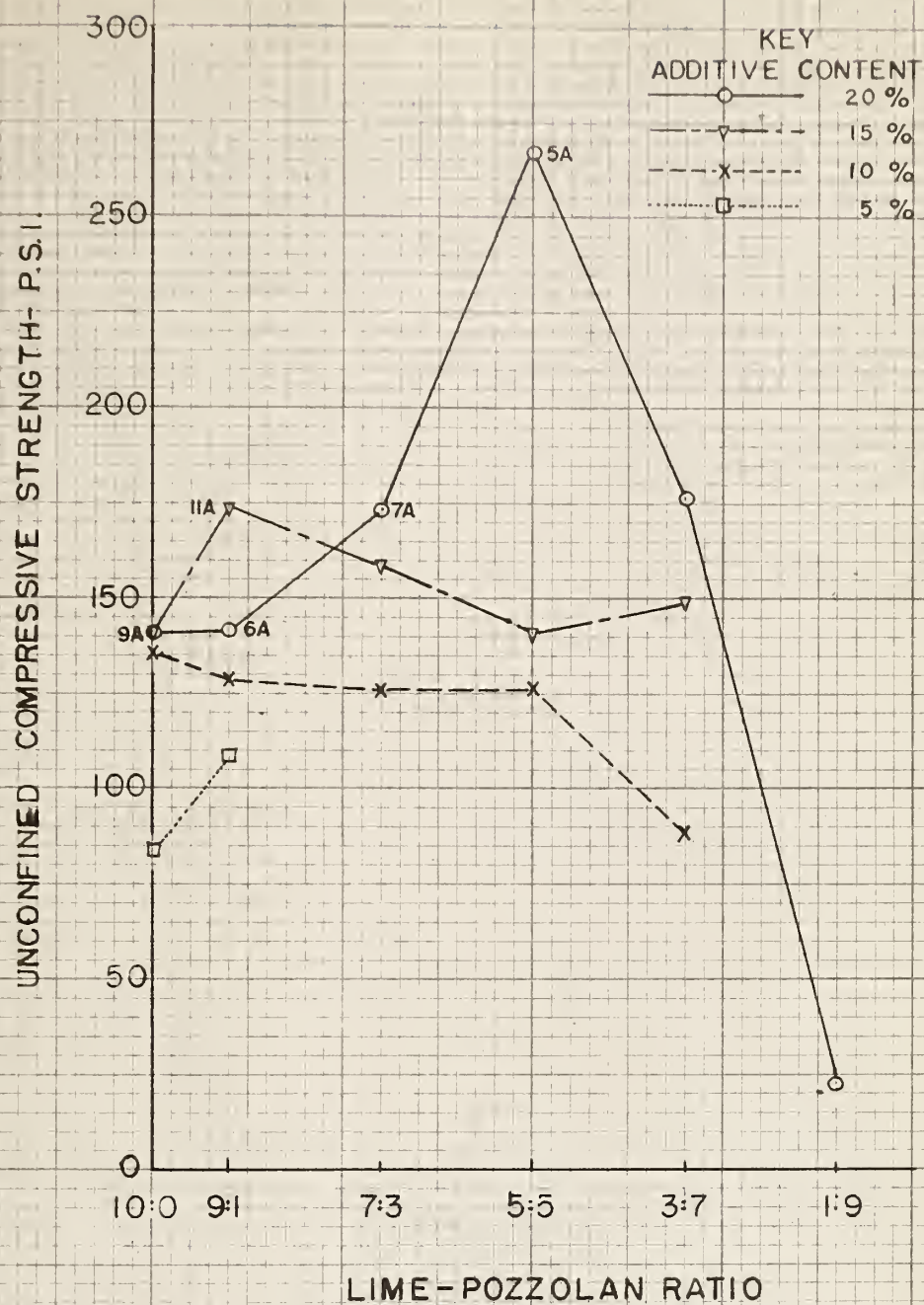
The strength values of mixture 5A seemed to be unusually high, while the strengths of mixtures 6A and 7A were quite low when compared with their counterparts produced with the higher compactive energy.

One concludes from examination of Figures 5 and 6 that the mixtures 5A, 6A, 7A, 9A and 11A behaved in a contrary manner.

#### The Lime-Pozzolan Ratio

Figures 7 and 8 present the same data as Figures 5 and 6, but the lime to pozzolan ratios are emphasized rather than the additive content. The lowest 28-day strengths, when samples of equal additive content were compared, were obtained from mixtures with a low lime content and a high proportion of pozzolan. Apparently, there was insufficient lime present in these mixtures to react with the available pozzolan. The pozzolan additive can be administered in smaller portions than those required for a non-pozzolan silt, since clay soils possess some inherent pozzolanic reactivity.





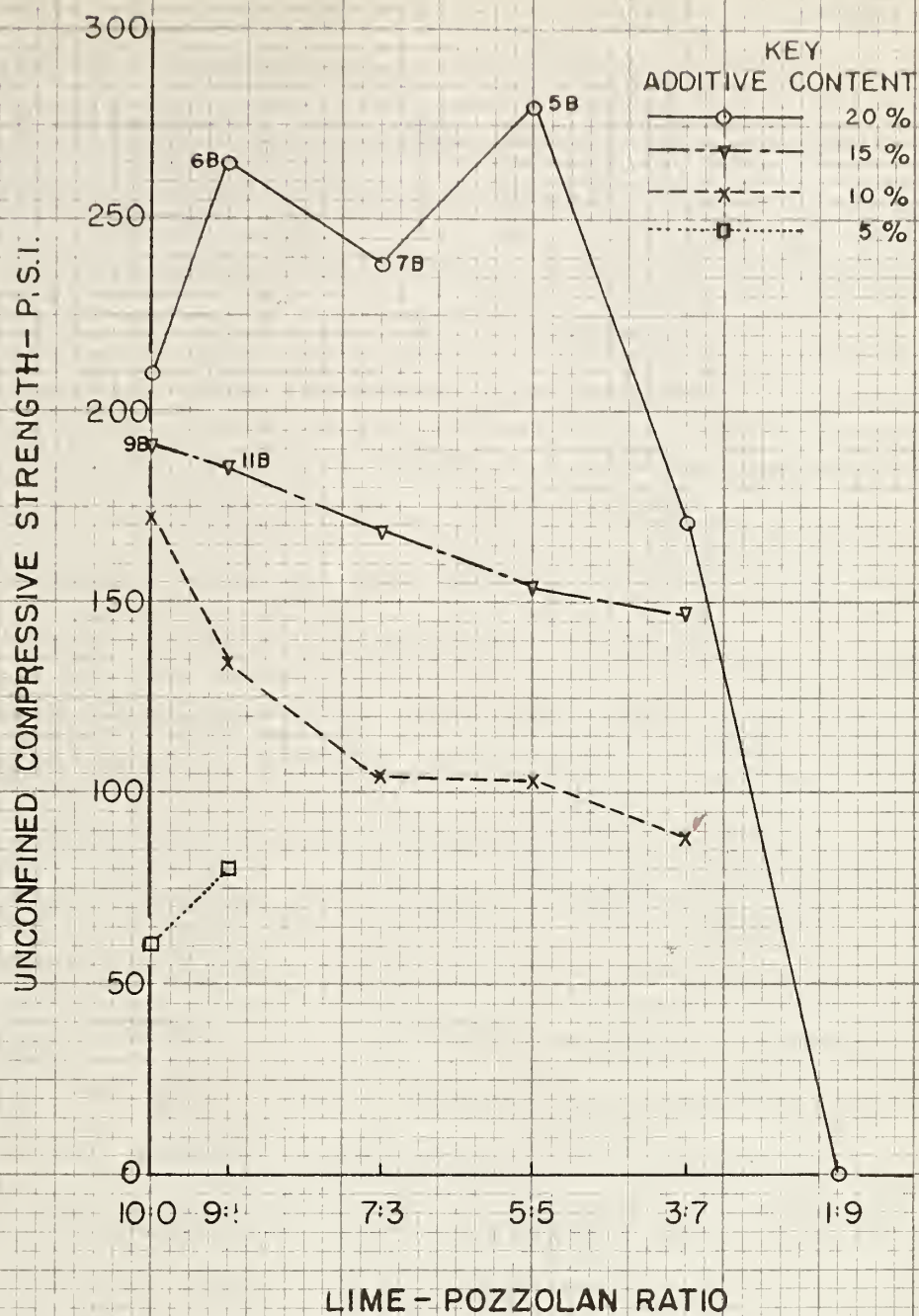
### UNCONFINED COMPRESSIVE STRENGTH VS LIME-POZZOLAN RATIO

5 BLOWS EACH FACE, 5 LB. WEIGHT  
CURING TIME, 28 DAYS; SOAKING TIME, 1 DAY

FIGURE 7







UNCONFINED COMPRESSIVE STRENGTH VS LIME-POZZOLAN RATIO

10 BLOWS EACH FACE, 10 LB. WEIGHT  
CURING TIME, 28 DAYS; SOAKING TIME 1 DAY

FIGURE 8



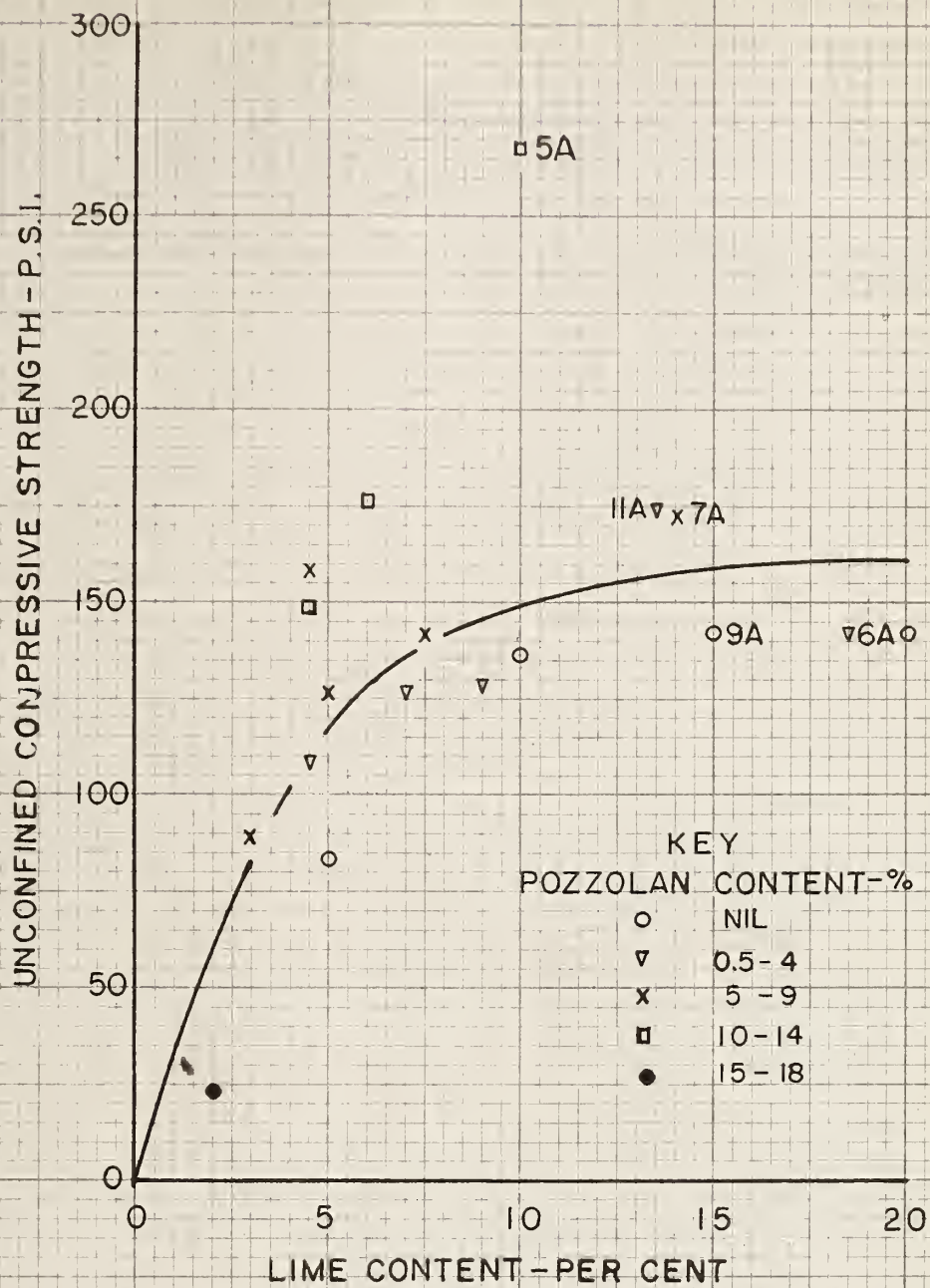
The unconfined compressive strength was plotted as a function of the lime content in Figures 9 and 10. The addition of lime generally increased the strength, whereas the proportion of pozzolan required was more complex. Large increases in strength were indicated if one added pozzolan in the amount of 10 to 14 per cent, to mixtures containing more than 10 per cent lime. However, when strength was plotted as a function of the pozzolan content, the results, which were not presented in this report, showed no correlation between the strength and the quantity of pozzolan, and the indications of high strengths could be attributed to the unusually high values of mixtures numbered 5A and 5B. A mixture containing 15 per cent lime and 15 per cent pozzolan would have been helpful in evaluating the existing data.

Upon reexamination, Figures 7 and 8 indicated higher strengths as more pozzolan was added to mixtures containing 5 per cent additive. It is unfortunate that mixtures 20 and 21 were not batched, because they may have produced strengths equal to those mixtures containing ten per cent additive.

The strength characteristics of the mixtures containing 5, 10 and 15 per cent additive seemed to conform to a common pattern for the results of both compactive efforts. The mixtures containing 20 per cent additive, in particular 5, 6 and 7, were not in harmony. Mixture 6A seemed to have a particularly low value of strength when compared to the relative position of mixture 6B in the Figures.







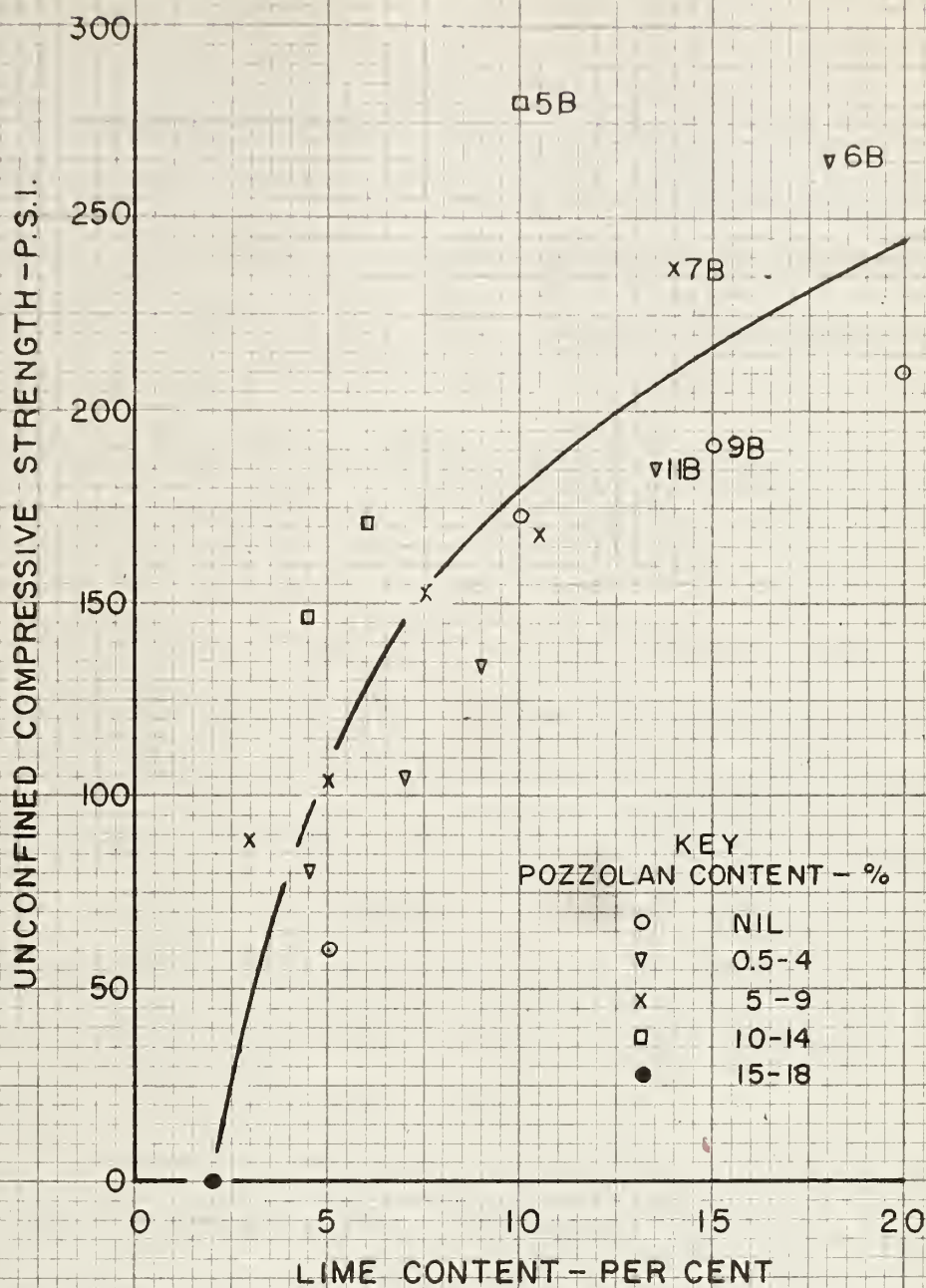
UNCONFINED COMPRESSIVE STRENGTH VS CONTENT OF LIME

5 BLOWS EACH FACE, 5 LB. WEIGHT  
CURING TIME 28 DAY; SOAKING TIME 1 DAY

FIGURE 9







UNCONFINED COMPRESSIVE STRENGTH VS CONTENT OF LIME

10 BLOWS EACH FACE, 10 LB. WEIGHT  
CURING TIME 28 DAYS; SOAKING TIME 1 DAY

FIGURE 10



### The Moisture Content and Dry Density When Moulded

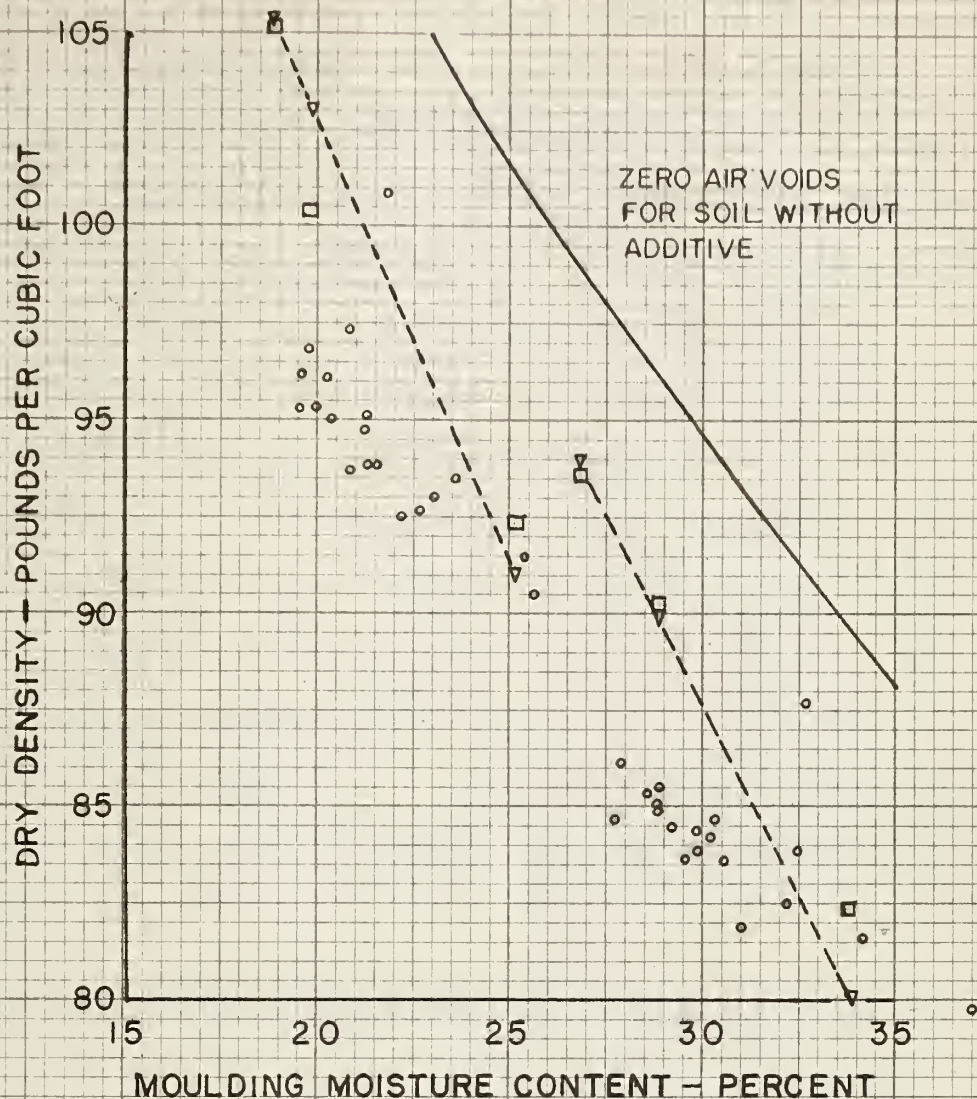
The moulding moisture contents used for the extremities of the design charts shown on Figures 2 and 4 were the optimum moisture contents capable of producing maximum dry densities. The optimum moisture contents of the intermediate mixtures were determined by interpolating among the values at the apices.

Figure 11 illustrates the fallacy in using this procedure for lengthy mixing and rotting periods. Even when the lime was added in amounts as low as 2 per cent by weight, the moisture requirements of the mixtures were considerably increased. If the moisture was not available, then the specimen was compacted on the dry side of the optimum moisture content and a reduced dry density resulted. The bottom extremity of the design curve in Figure 11 represents the moulding moisture content and dry density of a mixture containing 20 per cent lime. The uppermost point on the design curve represents a mixture containing 20 per cent pozzolan.

The points, which represent the actual densities and moulding moisture contents of the compacted specimens, fall in a band that has much less slope than the design curve. A more realistic design chart would contain experimentally determined values for mixtures 2, 5 and 8; with the other mix designs being computed by interpolating and extrapolating among the experimental values.







## KEY

- LOCI OF DENSITIES COMPUTED FOR MIXES SHOWN ON FIGS. 1 TO 4
- ▽ DENSITIES COMPUTED FOR MIXES AT EXTREMITIES OF FIGS. 1 TO 4
- OPTIMUM DENSITIES FOR MIXES AT EXTREMITIES OF FIGS. 1 TO 4  
(FROM APPENDIX B)
- DENSITIES OF STRENGTH AND DURABILITY SPECIMENS AS COMPACTED

MOISTURE - DENSITY RELATIONS AT TIME OF COMPACTION

ROTTING PERIOD, 24 HOURS

FIGURE 11





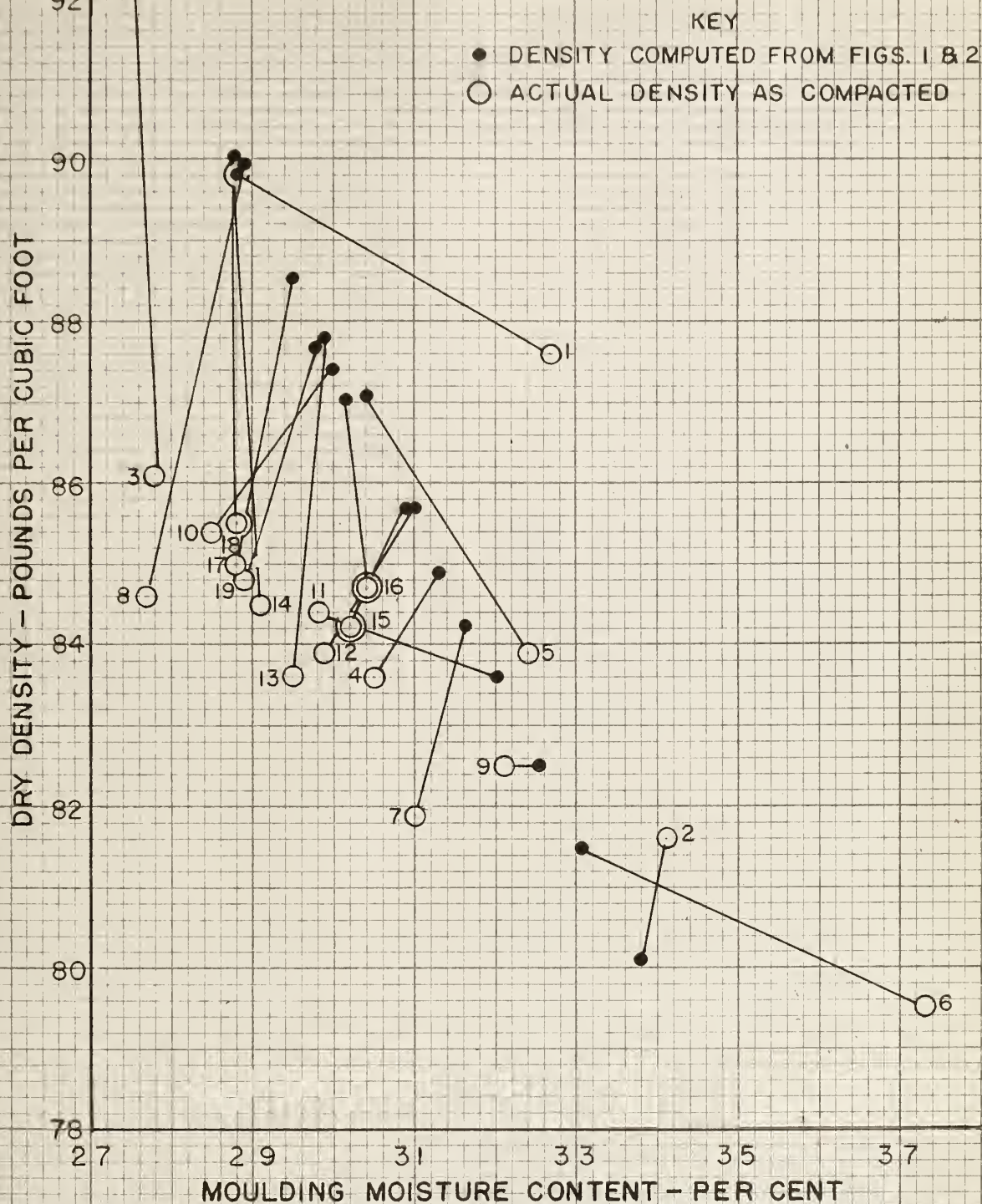
Closer examination of Figure 11 discloses the fact that the dry densities used for the design curves did not coincide with the optimum values obtained from the moisture-density tests. The discrepancy was not intended and was due to a mistake in the calculations, but it tended to increase the difference between the dry densities of the specimens as compacted and the optimum values for the same mixtures. It is very likely that many of the mixes, which did not conform to the general strength patterns, were compacted at moulding moisture contents other than optimum. The shifts in the moulding moisture contents and dry densities from the design curve are illustrated in Figures 12 and 13.

The following hypothesis is proposed between the unconfined compressive strength and shifts from the design curve of the moulding moisture content.

Hypothesis. Specimens compacted at moisture contents greater than those shown on the design curve yielded higher 28-day strengths than if the same specimens had been compacted at moisture contents that were less than the design values.

Consider Figure 13 first, because the shifts in density and moisture content were more regular for the specimens which received the higher compactive effort. Most of the specimens contained from  $\frac{1}{2}$  to 1 per cent less water than the designated quantity. The natural moisture content of the air-dried clay assumed for the purposes of calculations was 5 per cent, whereas various moisture content determinations ranged



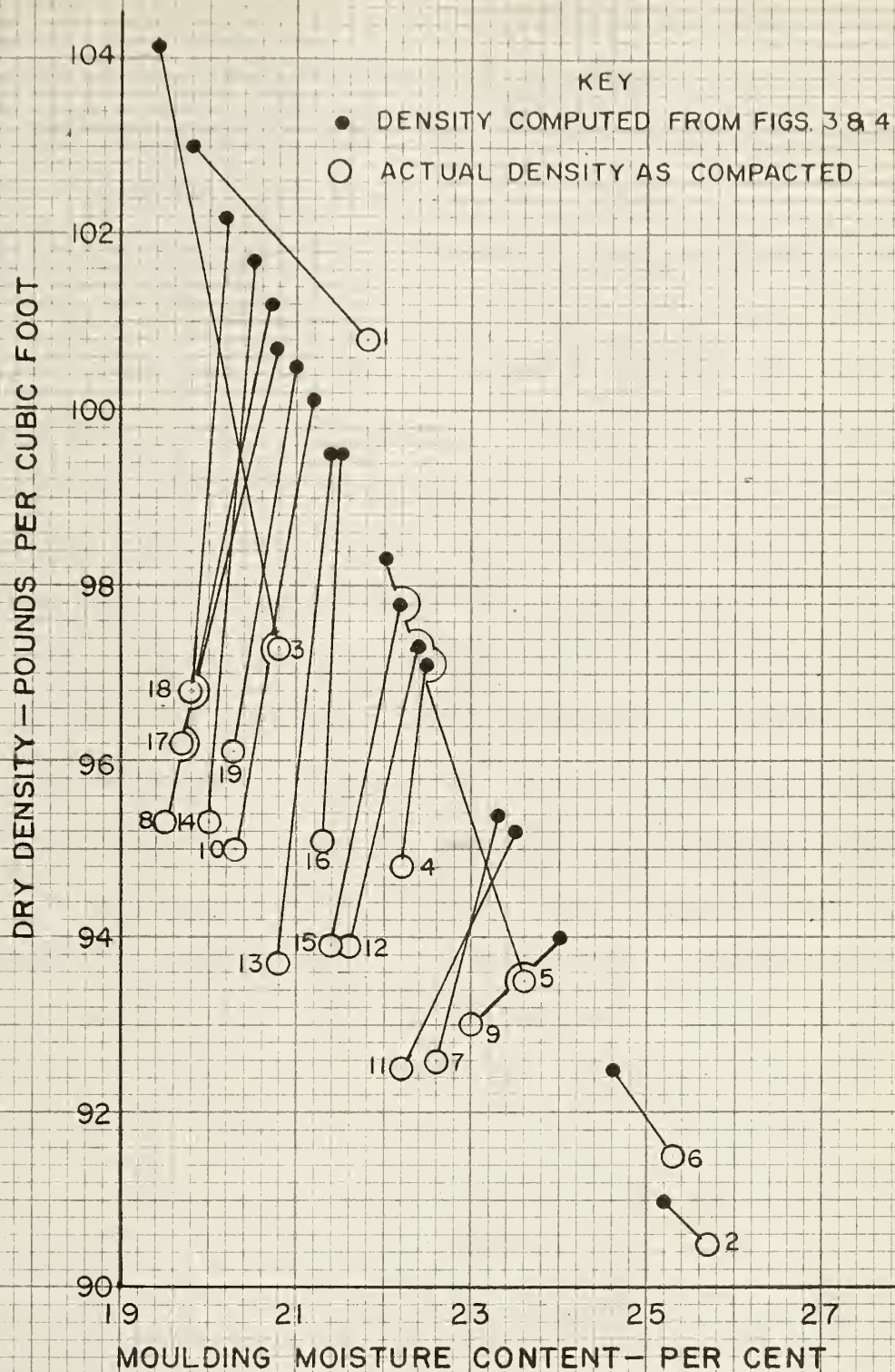


COMPARISON OF DESIGN VALUES WITH RESULTS AS COMPACTED  
FOR DRY DENSITY AND MOISTURE CONTENT  
5 BLOWS EACH FACE, 5 LB. WEIGHT

FIGURE 12







COMPARISON OF DESIGN VALUES WITH RESULTS AS COMPACTED  
FOR DRY DENSITY AND MOISTURE CONTENT  
10 BLOWS EACH FACE, 10 LB. WEIGHT

FIGURE 13





between  $4\frac{1}{2}$  to 5 per cent. Evaporation during the 1-hour mixing and weighing period and absorption of moisture by the dish and mixing spoon may have accounted for the remaining difference in moisture contents.

Notable exceptions to the general trend were mixtures 1B, 3B and 5B, in which the soil was assumed to contain four per cent moisture; and mixtures 2B and 6B, in which the natural moisture content was assumed to be 5 per cent. If the moulding content determinations were representative of the moisture content of the specimens, then it follows that an excess of water must have been mistakenly added to the mixtures. The excess of water could have placed the specimen near the optimum moisture content for strength. Thus specimens of mixtures 5B and 6B, which received an excess of mixing water were stronger than the specimens of mixture 7B, whose moisture content was less than the designed value. Although mixture 2B did not display strengths as high as mixtures 5B and 6B it may have had a higher value than if the moisture content had shifted to the dry side of the design curve. This show of higher strength may explain why the strength of the specimens containing lime only, and compacted at the higher density, did not show an optimum strength at a point between the 15 and 20 per cent additive contents in the manner of the specimens compacted at the lesser density.

If one compares the data from Figures 12 and 7, a definite correlation can be made between the high strengths of



mixture 5A and the shift to the wet side of the moulding moisture content.

In the case of mixture 6A, a large shift of the moulding moisture content to the wet side probably caused a loss in strength. The magnitude of the shift may have been sufficient to cause the specimens to be compacted on the wet side of the optimum moisture content for strength. Batch 6A must have accidentally received an excess of mixing water because it was observed, when compacting the specimens, that the soil was on the wet side of the optimum moisture content for density. Thus, although the results of mixture 6A did not agree with the hypothesis, they can still be logically explained.

The results of the strength tests of specimens of mixture 11A belie the hypothesis, because a definite shift to the dry side of the design value seemed to result in strengths that were higher than expected. It is possible that the moisture content determination of mixture 11A was in error or was unrepresentative of the compacted specimens, but such a supposition is pure conjecture and is brought into the argument merely to support the hypothesis.

While Figure 12 and 13 are still under consideration, attention is drawn to the fact that the compacted dry densities of the samples of mixes 2A and 2B were greater than the design values. The actual dry densities, however, were still less than the optimum values shown on Figure 11.



### Rotting Time

In this investigation the specimens compacted at optimum dry density were assumed to be at optimum values of strength as well. Mateos and Davidson (1960) pointed out the changes that may occur to the optimum moisture content when rotting of a mixture is permitted. They also declared that the moulding moisture content may be different for optimum strength values and for optimum dry density.

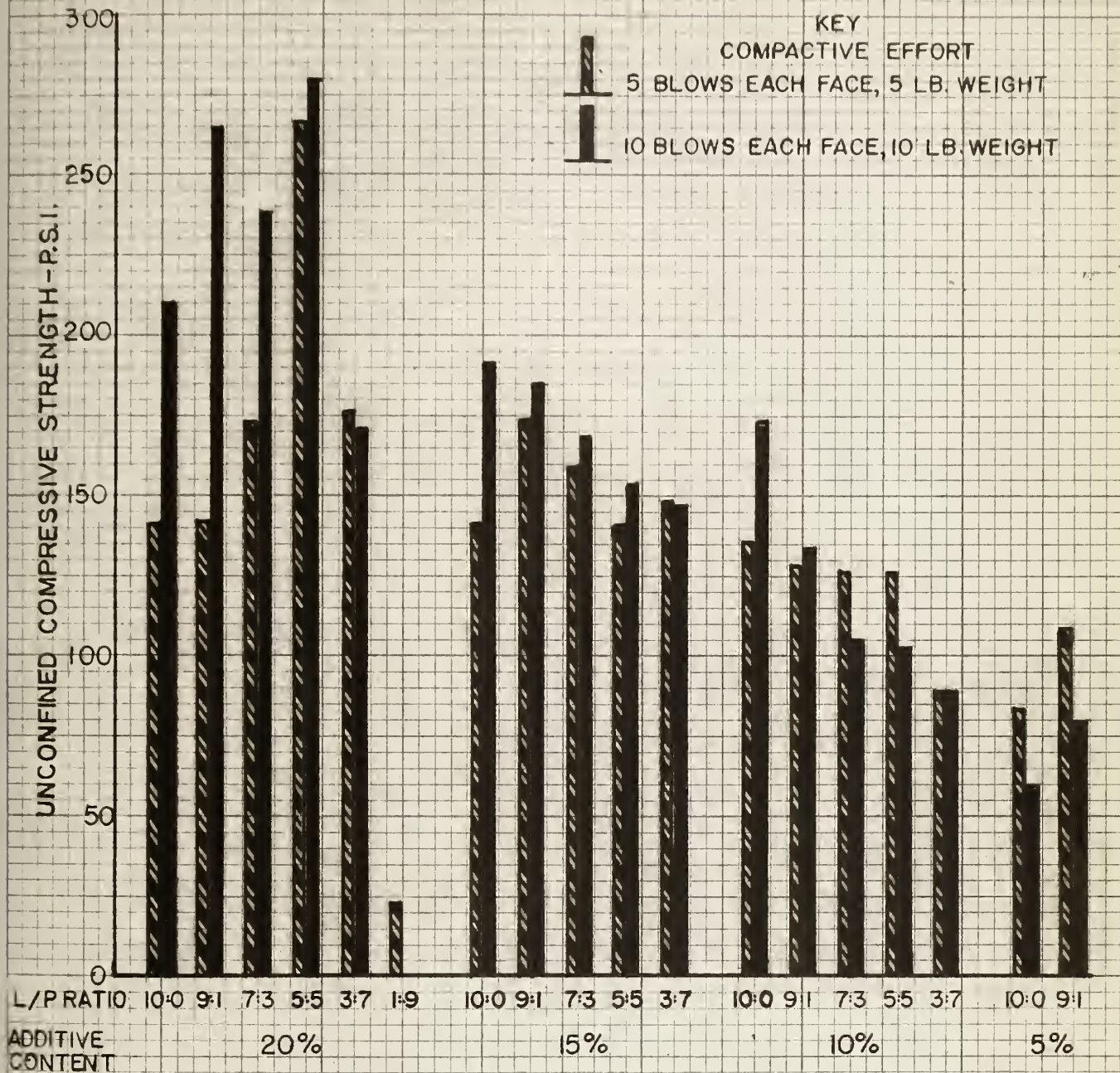
If this phenomenon was true for the clay-lime-pozzolan mixtures of this investigation, then this fact, coupled with the deviations experienced in the tests, would lead one to conclude that several and perhaps all of the values of strength obtained in this investigation were not maxima. The mixes which indicated the highest strengths in this investigation may not be the best mixes if one obtained the optimum strength for each mixture. Thus our examination of the results must be restricted to observing trends rather than studying the results in detail.

### Compactive Effort

A study of Figure 14 shows that increased compactive effort improved the 28-day unconfined compressive strength of mixtures containing more than 15 per cent additive, which had lime-pozzolan ratios greater than unity. For the remaining mixes, the compactive effort seemed to have little beneficial effect and in some cases the additional compaction seemed to be detrimental. The literature reviewed in section II.4 was







UNCONFINED COMPRESSIVE STRENGTH VS COMPACTIVE EFFORT  
 FOR VARIOUS LIME AND POZZOLAN CONTENTS  
 CURING TIME 28 DAYS; SOAKING TIME 1 DAY

FIGURE 14



unanimous in stating that increased compactive effort resulted in increased strengths. The fact that the results of this investigation did not agree with the published literature on this topic is another indication that the values being collated in Figure 14 were not optimum values and therefore could not be fairly compared.

#### The Voids-Cement Ratio

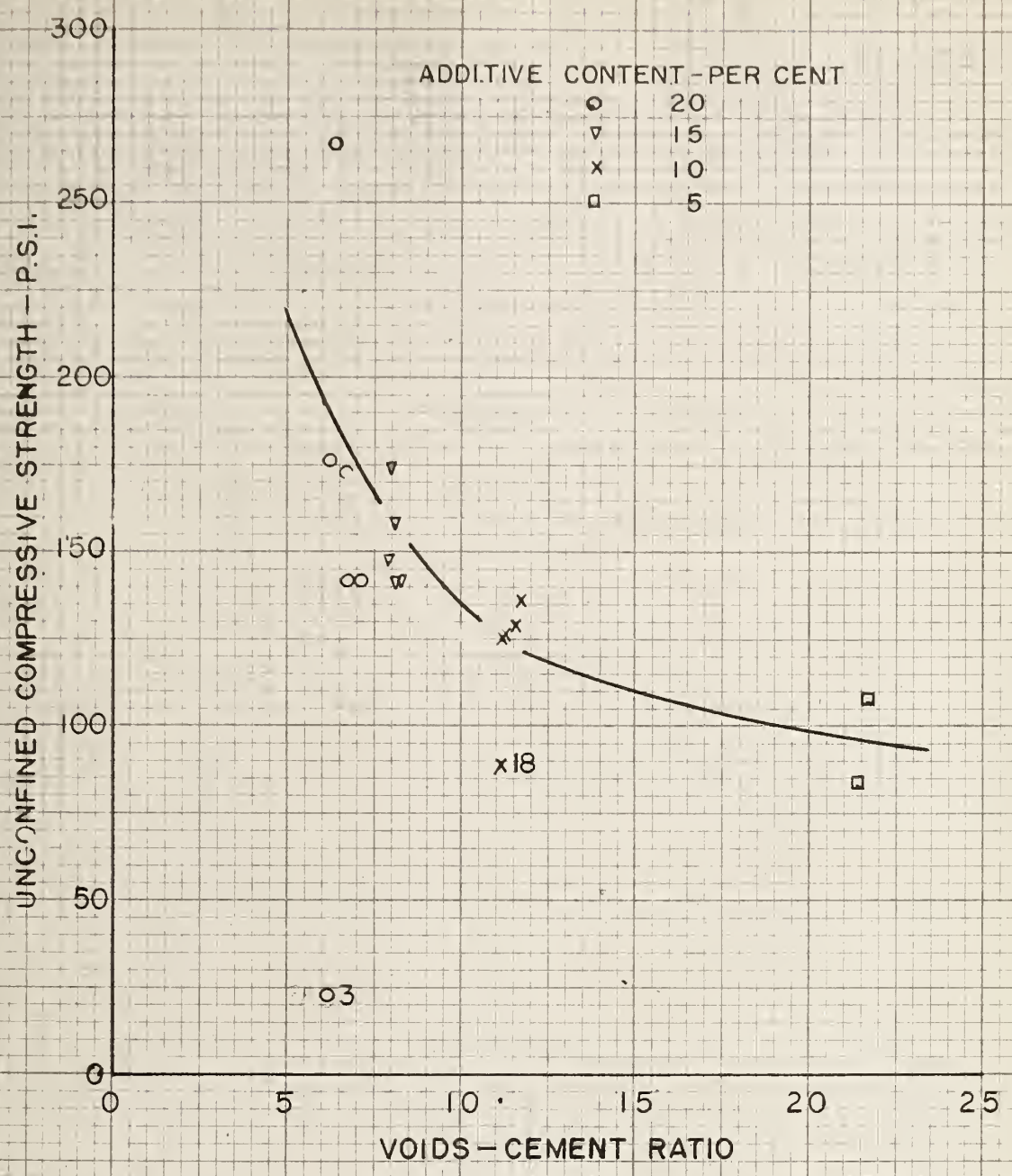
The 28-day unconfined compressive strength was plotted as a function of the voids-cement ratio in Figures 15 through 18. Best fit curves indicated that lower void-cement ratios denoted higher unconfined compressive strengths. When one considers the possible error that may be involved in the data, it is encouraging to see such good fits for the curves.

The most important variable in the voids-cement ratio was the additive content. The effect on the voids-cement ratio was not as great for the large additive contents as it was for the low values. From Figures 15 and 16, it appears that for the higher additive contents the reduction in the voids-cement ratio comes from an increase in the cement content rather than a reduction in the volume of voids.

Increasing the compactive effort decreased the voids-cement ratio by decreasing the volume of the voids. The moulding moisture content undoubtedly is another variable which affects the void-cement ratio since changes in the moulding moisture can vary the volume of the voids for any given compactive effort.







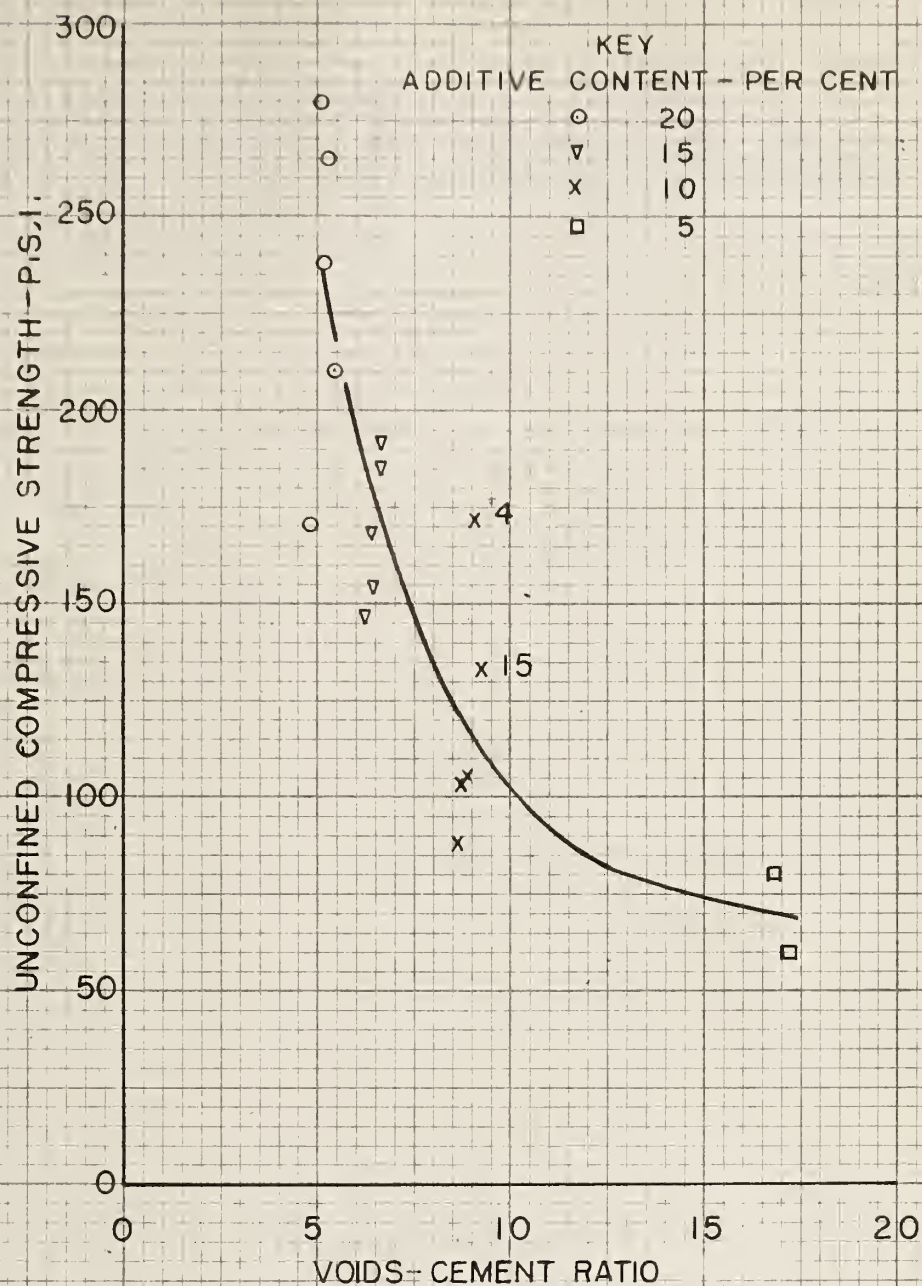
UNCONFINED COMPRESSIVE STRENGTH VS VOIDS-CEMENT RATIO  
FOR EACH ADDITIVE CONTENT

5 BLOWS EACH FACE, 5 LB. WEIGHT  
CURING TIME 28 DAYS; SOAKING TIME 1 DAY

FIGURE 15





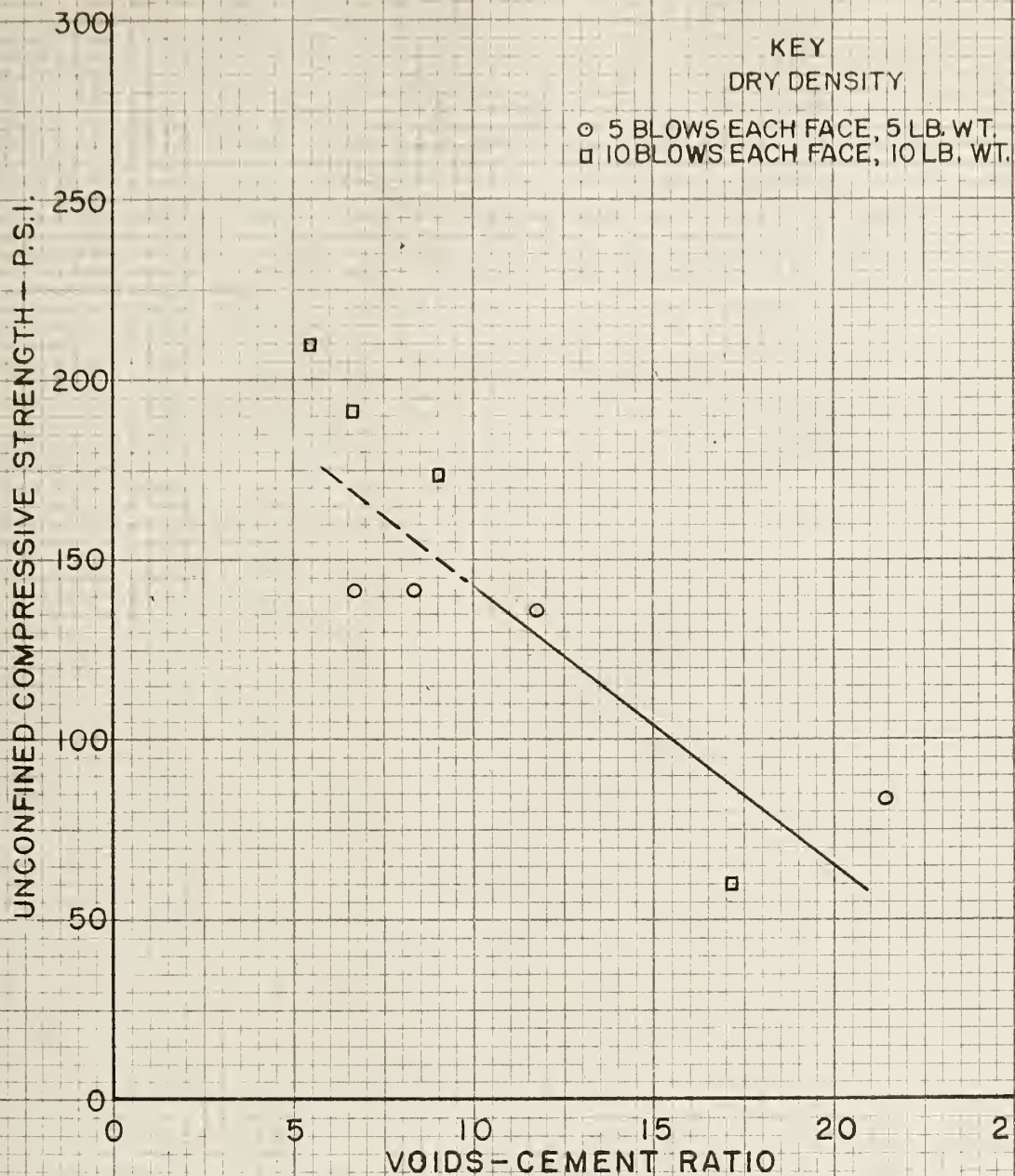


UNCONFINED COMPRESSIVE STRENGTH VS VOIDS-CEMENT RATIO  
FOR EACH ADDITIVE CONTENT

10 BLOWS EACH FACE, 10 LB. WEIGHT  
CURING TIME 28 DAYS; SOAKING TIME 1 DAY

FIGURE 16



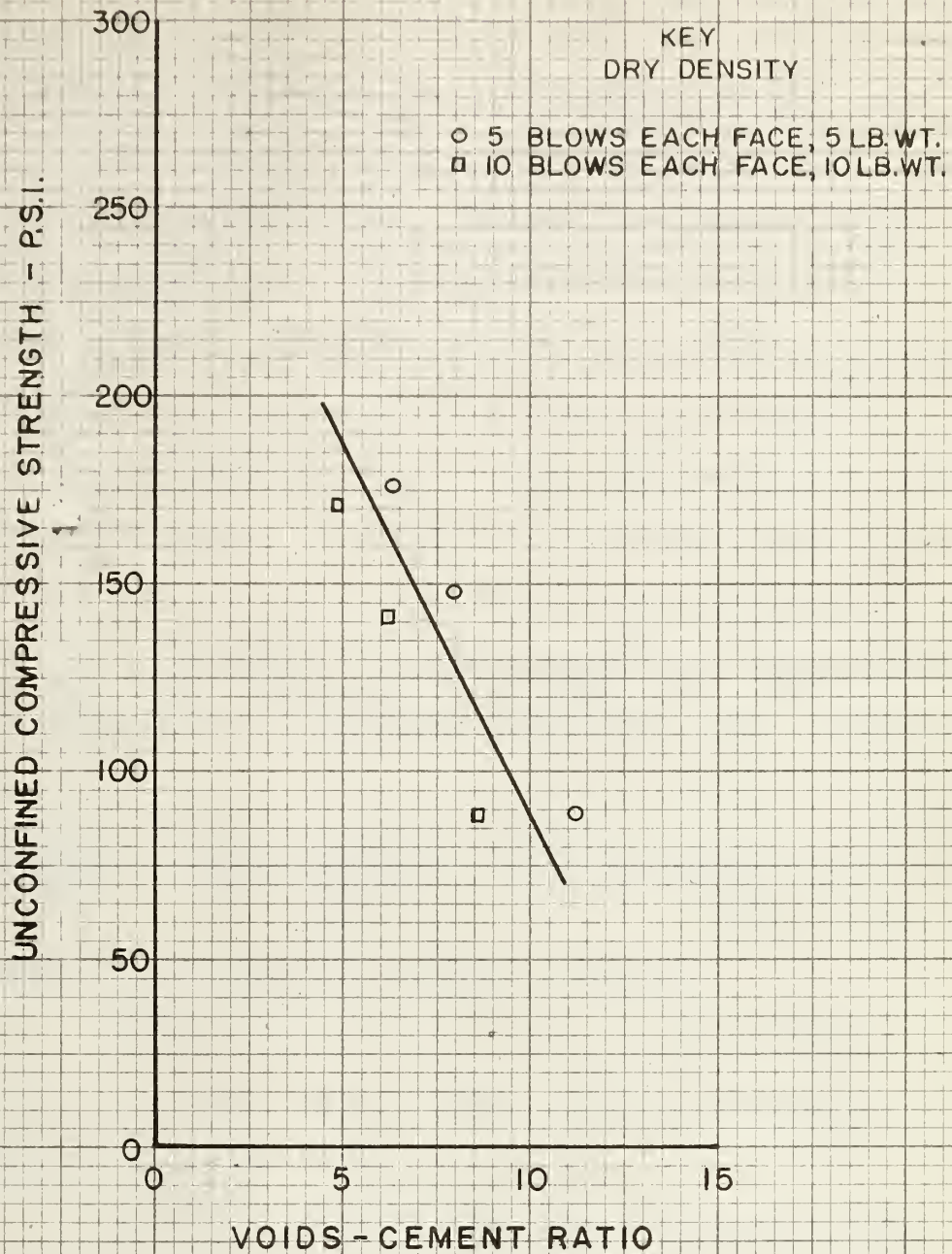


UNCONFINED COMPRESSIVE STRENGTH VS VOIDS-CEMENT RATIO  
FOR A LIME-POZZOLAN RATIO OF 10:0  
CURING TIME 28 DAYS; SOAKING TIME 1 DAY

FIGURE 17







UNCONFINED COMPRESSIVE STRENGTH VS VOIDS-CEMENT RATIO  
FOR A LIME-POZZOLAN RATIO OF 3:7  
CURING TIME 28 DAYS; SOAKING TIME 1 DAY

FIGURE 18





The lime-pozzolan ratio showed no effect upon the voids-cement ratio since an average value was used for the specific gravity of the additive. Specific gravity tests which were run on the lime and the pozzolan resulted in determinations of 2.76 and 2.73 respectively. Boynton and Jander (1952) list the specific gravities of calcitic hydrated limes in the range between 2.3 and 2.4. However, higher values can be obtained depending upon the type of limestone and the temperature of burning. If the specific gravity were 2.28 instead of 2.76 the voids-cement ratio of mixture 7A would have been reduced from 6.64 to 4.94 and that of 8A would have been reduced from 6.24 to 4.11. The formula for calculating the voids-cement ratio for any values of specific gravity of lime, pozzolan and soil is

$$\frac{\text{Volume Voids}}{\text{Volume Cement}} = \frac{\gamma_w \cdot G_L (1 + p + q)}{\gamma_D (p + \frac{G_p}{G_L} \cdot q)} - \left[ 1 + \frac{G_L}{G_s (p + \frac{G_p}{G_L} \cdot q)} \right]$$

where  $\gamma_w$  is the unit weight of water,

$\gamma_D$  is the dry unit weight of the mixture

p is the ratio of the weight of pozzolan to the weight of dry soil

q is the ratio of the weight of lime to the weight of dry soil

$G_s$  is the specific gravity of the soil

$G_p$  is the specific gravity of the pozzolan

$G_L$  is the specific gravity of the lime.

Thus the voids-cement ratio is also dependent upon the



type of soil, pozzolan, and lime; and to slight extent upon the lime-pozzolan ratio.

The voids-cement ratio measure physical properties only and cannot indicate the pozzolanic activity of the components. Inasmuch as the physical properties of the mixtures affect the strength, the voids-cement ratio would be a good basis for a specification. For example, with the clay, lime and pozzolan used in this test, one could set a maximum value of void-cement ratio and expect to get a minimum unconfined compressive strength within those limits, provided sufficient lime was included to overcome the lime fixation capacity of the soil. If one examines Figures 15 and 16, however, they will notice that the voids-cement ratio could not be used to pick the most efficient mixture of lime, soil and pozzolan.

Figures 17 and 18 show the effect of the lime to pozzolan ratio upon the relationship between the unconfined compressive strength and the voids-cement ratio. In Figure 17, the apparent optimum lime content for the clay soil compacted at the lesser density is evident and a curve through the four points would not follow the pattern indicated by Figures 15 and 16. In Figure 18, the lowest strength values were probably due to the low lime contents of 2 and 3 per cent, for mixtures 4 and 15 respectively. These lime contents were probably just above the lime-fixation capacity of the soil, and thus could not provide higher strengths even at low void-cement ratios.



### The Effect of Curing and Soaking

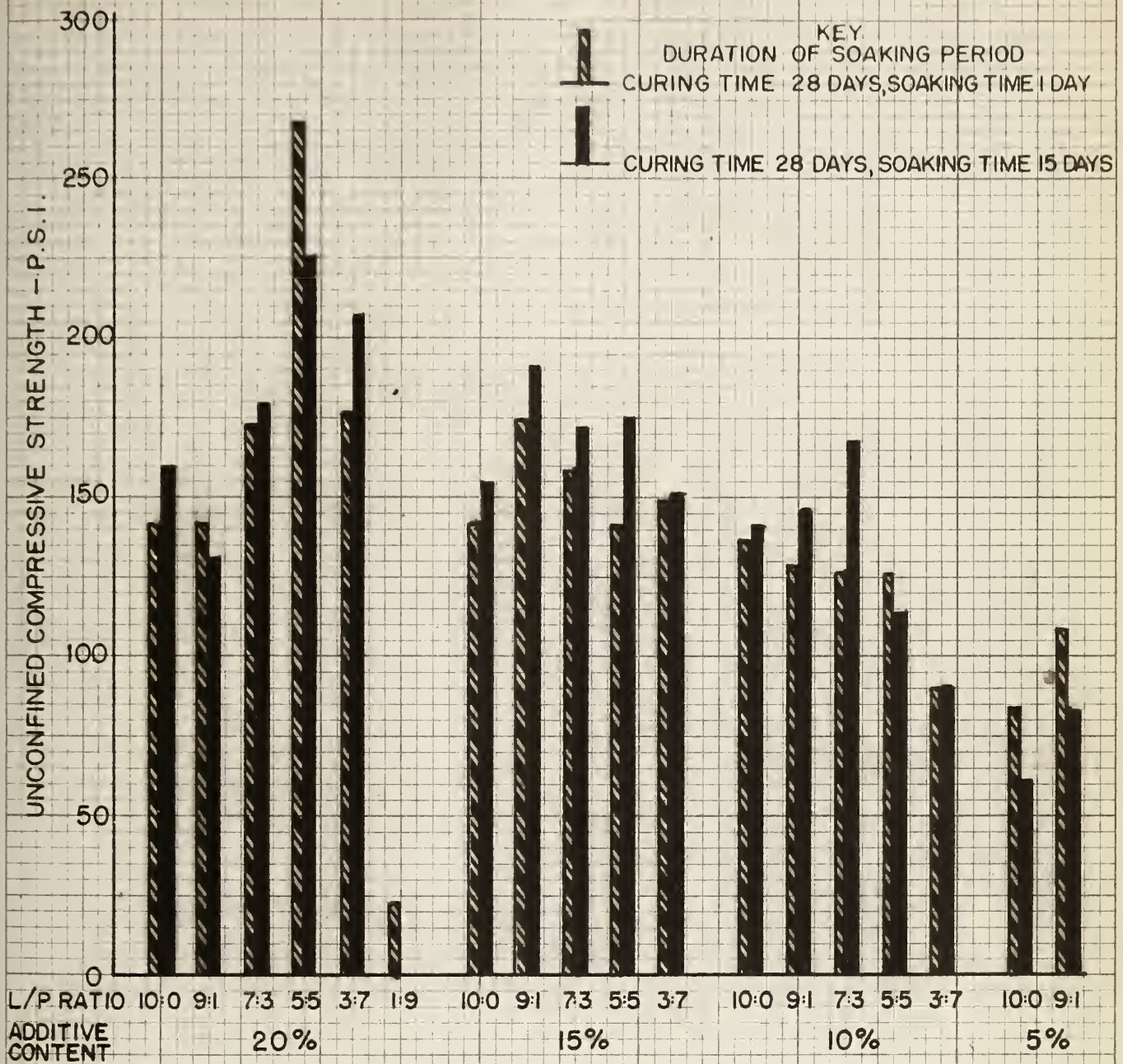
Figures 19 and 20 show the effect of prolonged wetting upon the strength of lime and pozzolan mixtures. The Figures can be compared without regarding the effects of the moulding moisture contents on the unconfined compressive strengths because the specimens for the 28-day strengths and for the freeze-thaw control tests were identical up to the end of the curing period.

The specimens compacted at the lesser density generally gained in strength during the soaking period of 15 days. The cylinders compacted with the higher compactive effort generally lost strength during the soaking period of 8 days. There were exceptions to both generalities and these exceptions did not seem to follow any general pattern. The largest numerical loss in strength was 63 p.s.i. and the largest proportional decreases were 100 per cent in the case of mix 3A and 36 per cent for mix 19B. The largest numerical increase in strength was 42 p.s.i. and the largest proportional increase in strength was 32 per cent. Although these variations in strength were considerable, they also seemed to be unpredictable.

The specimens compacted to the higher density that disintegrated when soaked seemed to break up more rapidly than did the companion specimens of the lesser density. The disintegration appeared to be a combination of slaking with the escape of entrapped air bubbles from the interior of the specimens. Thus the mixtures compacted at the higher density may not have





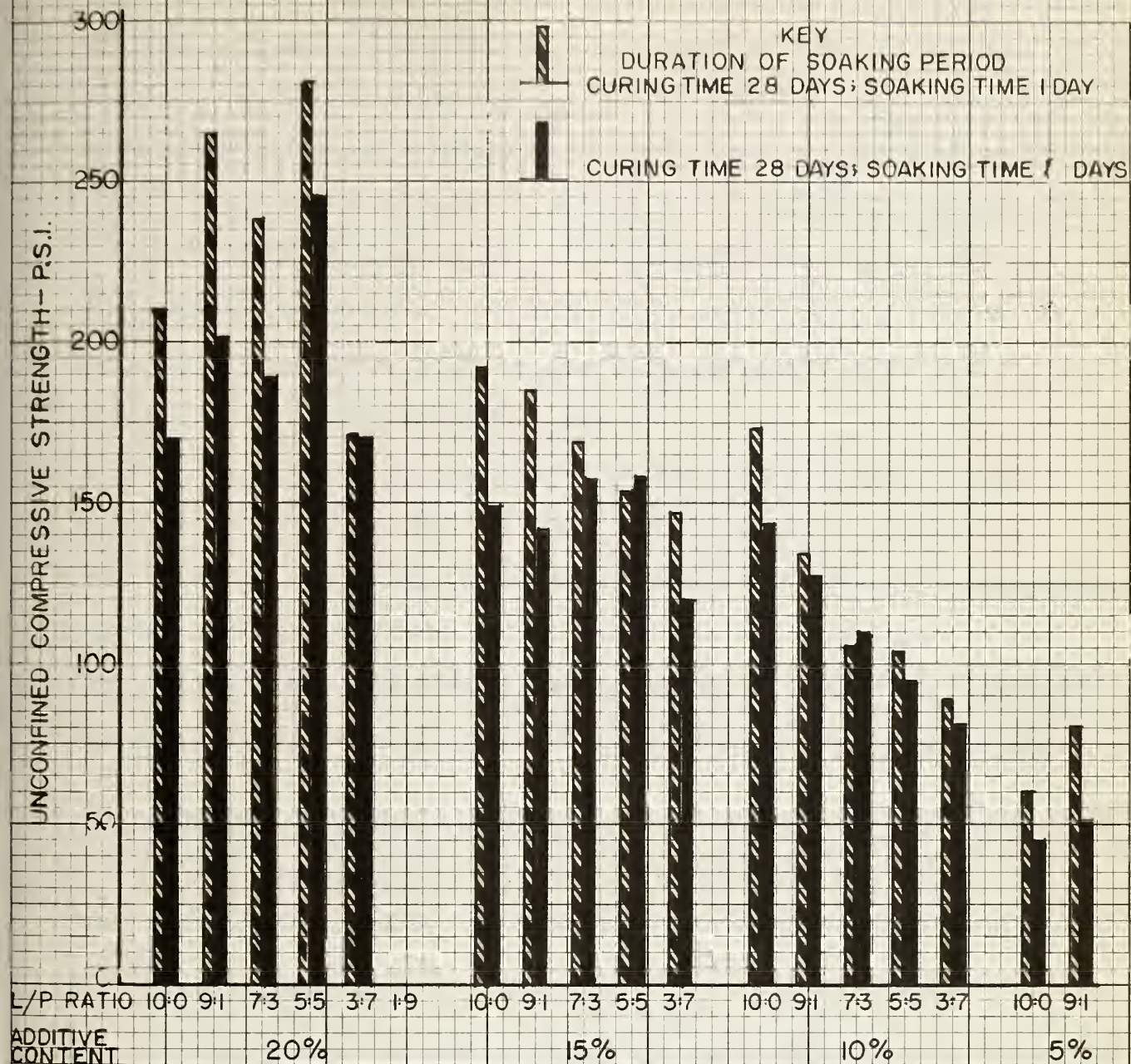


UNCONFINED COMPRESSIVE STRENGTH VS ELAPSED TIMES OF SOAKING  
FOR VARIOUS LIME AND POZZOLAN CONTENTS  
5 BLOWS EACH FACE, 5 LB. WEIGHT

FIGURE 19







**UNCONFINED COMPRESSIVE STRENGTH VS ELAPSED TIMES OF SOAKING  
 FOR VARIOUS LIME AND POZZOLAN CONTENTS  
 10 BLOWS EACH FACE, 10 LB. WEIGHT**



possessed as high degree of saturation during the curing period as the specimens compacted at the lesser compactive effort. When the specimens were immersed, they swelled considerably within 24 hours and continued to swell over a period of several days. A detailed study of the data to correlate the change in volume of the air and water in the voids with changes in the unconfined compressive strength might result in explaining why certain specimens gained or lost strength when subjected to prolonged periods of immersion.

It is possible that the specimens lose strength upon swelling, but once the swelling is completed the strength may be built up to values that surpassed the strengths possessed at the end of one day's immersion. Thus the specimens soaked for 15 days showed an increase in strength, but those soaked for 8 days were still less than the strengths after 1 day's immersion.

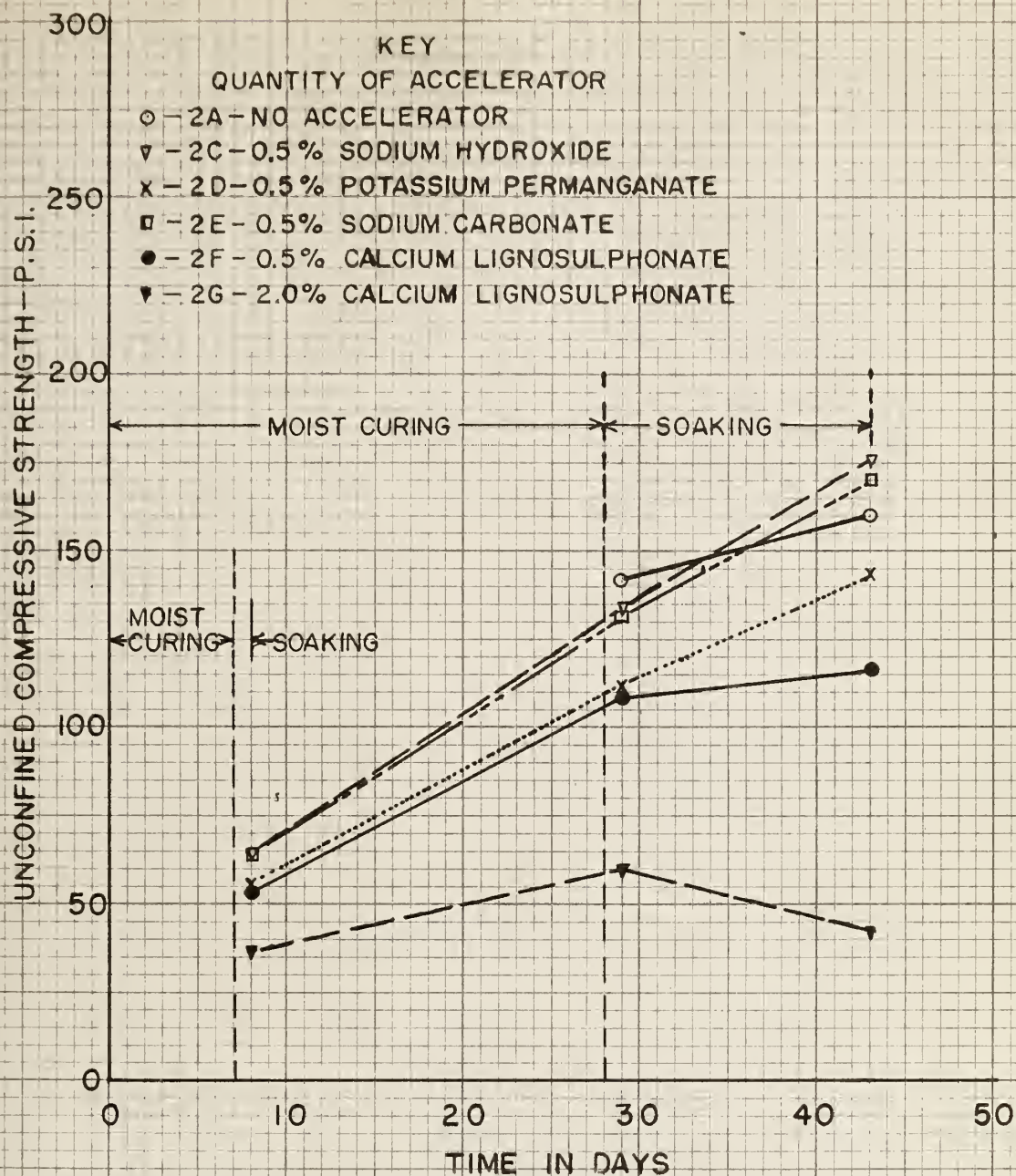
### Accelerators

The results of the tests using strength accelerators are tabulated in Table VII and displayed graphically in Figures 21 and 22.

Type of accelerator. None of the specimens compacted from mixtures containing accelerators possessed as high 28-day strengths as the untreated specimens. The samples containing sodium carbonate were the best of the lot, giving 28-day strengths only slightly less than the strengths of the control samples. The potassium permanganate caused a drop in the 28-day strength of







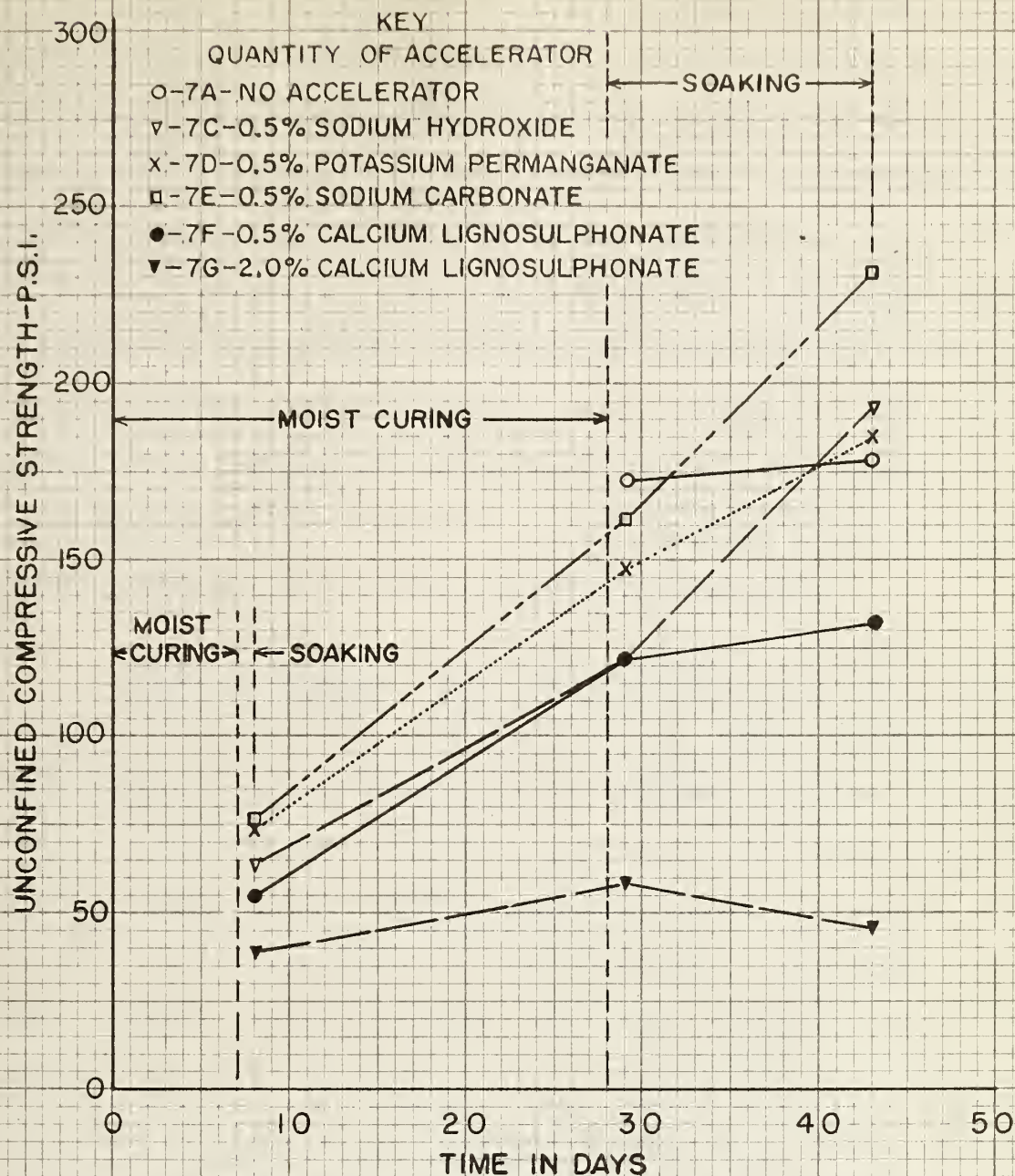
UNCONFINED COMPRESSIVE STRENGTH  
VS  
ELAPSED TIMES OF CURING AND SOAKING  
FOR  
SPECIMENS TREATED WITH ACCELERATORS  
LIME-POZZOLAN RATIO-10:0

5 BLOWS EACH FACE, 5 LB. WEIGHT  
LIME-POZZOLAN CONTENT OF 20 PER CENT

FIGURE 21







**UNCONFINED COMPRESSIVE STRENGTH  
VS  
ELAPSED TIMES OF CURING AND SOAKING  
FOR  
SPECIMENS TREATED WITH ACCELERATORS**

LIME-POZZOLAN RATIO, 7:3  
5 BLOWS EACH FACE, 5 LB. WEIGHT  
LIME-POZZOLAN CONTENT OF 20 PER CENT

FIGURE 22



almost 30 p.s.i. The effect of the sodium hydroxide was unpredictable; showing a loss in the 28-day strength of only 9 p.s.i. for the mixtures containing 20 per cent lime; and displaying a drop of 51 p.s.i. when added to the mixtures containing 14 per cent lime and 6 per cent pozzolan.

The calcium lignosulphonate caused substantial reductions in the 28-day strengths of both mixtures; the worst results being shown by the mixtures containing 2 per cent lignosulphonate. Adding the lignosulphonate in the amount of 2 per cent prevented the formation of the large aggregates which were common to the other lime treated specimens.

Time of curing. The 7-day-strength specimens were not the same quantity or quality as the other cylinders, so the results can be relied upon to indicate minimum strengths only. However, the 7-day-strength tests did predict the respective efficiencies of the accelerators at the end of 28 days curing. With the exception of the mixture containing 2 per cent calcium lignosulphonate, the 7-day strengths were approximately one half the 28-day value. The specimens containing 2 per cent calcium lignosulphonate gained very little strength in the intervening period of 21 days.

7-day-strength specimens were not produced for mixtures 2A and 7A. It should be noted that moulding moisture content variations did not affect the collation of the 7-day and 28-day values because the specimens were identical at the beginning of the curing period.





Time of soaking. Whereas the untreated specimens gained little strength during the periods of prolonged soaking, the treated specimens continued to gain strength at the same rate or at a more rapid rate, than during the periods of moist curing. The preceding statement does not cover the special case of the mixtures treated with calcium lignosulphonates, which will be discussed in a paragraph at the end of this subsection.

To explain this phenomena of accelerated strengths through soaking, consider the relative solubilities in water of the hydroxides of sodium and calcium. Sodium hydroxide has a solubility of 52.2 grams per 100 grams of solution at a temperature of 20C.<sup>1</sup> Calcium oxide is dissolved at the rate of 1.17 grams per liter at the same temperature but when sodium hydroxide in the amount of 2.666 grams per liter is present in the solution, the solubility of the calcium oxide is reduced to 0.39 grams per liter.<sup>2</sup>

When the specimens were cured in the presence of the moulding water only, the sodium hydroxide was probably completely dissolved, but very little of the calcium hydroxide was in solution. There were fewer free calcium ions to react with the silicates than if the sodium hydroxide was not present in the

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<sup>1</sup>From Thorpe's Dictionary of Applied Chemistry, Vol. X, (1950).

<sup>2</sup>From Thorpe's Dictionary of Applied Chemistry, Vol. II, (1938).

The American Medical Association is a non-profit corporation organized for the purpose of promoting the interests of the medical profession and the public. It is composed of the members of the American Medical Association, who are organized into local, state, and national associations. The Association is organized into a hierarchy of committees and boards, which are responsible for the management of the Association's affairs. The Association's primary concern is the promotion of the highest standards of medical practice and the improvement of the medical profession. It does this by publishing the Journal of the American Medical Association, which is one of the most authoritative sources of medical information in the world. The Association also sponsors a variety of educational programs and conferences, and it works to improve the medical profession's relationship with the public.

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water. Thus the pozzolanic reaction was retarded, and at the end of 28 days of moist curing the accelerated specimens had lower strengths than the untreated specimens.

The sodium, in the meantime, released the silicates from the clay minerals and pozzolan and formed soluble sodium silicate. Since the lime depressed the solubility of sodium silicate, the small concentration of lime allowed a larger concentration of sodium silicates in the water.<sup>1</sup>

Prolonged soaking, diluted the sodium concentration and subsequently allowed more calcium hydroxide to go into solution. The pozzolanic reaction between the lime and the well distributed sodium silicate gave higher strengths than if the sodium had not made the silicate so readily available. Thus the treated specimens gained strength much more rapidly than the untreated specimens.

Although the above hypothesis explained the results of these tests, it cannot be applied to the results found by other investigators, in which the 28-day strengths were increased by the use of accelerators.

Perhaps both the treated and untreated specimens lost strength when they increased in volume, and regained it again after the swelling had ceased. The presence of accelerators may have speeded up the swelling process and thus at the end of 24 hours the treated samples were weaker than the control

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<sup>1</sup>From Thorpe's Dictionary of Applied Chemistry, Vol. X (1950). See Sodium Silicate.



specimens. The accelerators may also have speeded up the cementing action after the swelling had ceased giving higher strengths at the end of 15 days of immersion.

In comparing the results of various accelerators, one should be reminded of the results published by Davidson and others (1960). Each accelerator showed different results dependent upon the concentration of the accelerator. In order to judge the effectiveness of each chemical, mixtures should have been produced in which the concentration of accelerator was varied from  $\frac{1}{4}$  per cent up to 3 per cent. It is also unfair to judge the effectiveness of accelerators from their results with only two clay-lime-pozzolan mixtures.

The addition of the accelerators had little effect on the swelling characteristics of the specimens, with the exception of those treated with calcium lignosulphonate. Samples containing 2 per cent calcium lignosulphonate swelled so little in 24 hours that it is doubtful if the center of the specimens were saturated when tested. Thus part of the strength at twenty-eight days for the mixtures 2G and 7G could be attributed to surface tension between the soil particles.

## V.2 SUMMARY OF SIGNIFICANT RESULTS OF STRENGTH TESTS

The moulding moisture content of the samples varied considerably from the optimum moisture contents which would give maximum dry densities. Therefore the highest possible





strengths were not achieved for an undetermined number of the mixtures. The variability from the optimum strengths made detailed comparison of the results impossible, but the following general trends were evident.

1. Unconfined compressive strengths of more than 250 p.s.i. can be achieved from clay-lime-pozzolan mixtures which have undergone 24 hours of rotting, 28 days of moist curing at normal temperatures, and 24 hours of complete immersion in water.

2. The unconfined compressive strength can be increased by increasing the amount of additive.

3. The best unconfined compressive strengths were obtained when the lime to pozzolan ratio exceeded unity.

4. The unconfined compressive strength is a function of the voids-cement ratio, but the voids-cement ratio cannot indicate the pozzolanic reactivity of the various components: clay, lime and pozzolan.

5. Prolonged soaking of the specimens either increased or decreased the unconfined compressive strength, but no pattern was evident from which one could predict which mixtures would show decreased strengths when immersed for long periods.

6. The specimens produced from mixtures which contained sodium carbonate, sodium hydroxide and potassium permanaganate continued to gain strength at a constant or accelerated rate when soaked for period of 15 days. Specimens



which did not contain accelerators showed only slight increase during the same period.

7. Calcium lignosulphonate appeared to have a detrimental effect upon the strengths of the stabilized mixtures.

### V.3 RESULTS OF DURABILITY TESTS

Many of the shortcomings discovered in the freeze-thaw procedures were described in chapter IV. The results of the strength tests on the freeze-thaw specimens are presented in Tables V and VI, while the results of the freeze-thaw control samples are displayed in Tables III and IV. The number of specimens extruded intact from the freeze-thaw moulds and the number of extruded specimens judged fit to test after 24 hours immersion are listed in tabular form for the mixtures.

#### Specimens Compacted With 5 Blows Each Face, 5-lb. Weight

Mix Number	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
Extruded	-	2	3	4	4	6	6	6	4	6	6	5	3	6	6	6	6	6	4
Fit to Test	-	0	0	0	3	2	1	1	1	0	0	1	1	0	0	0	0	0	0

#### Specimens Compacted with 10 Blows Each Face, 10-lb. Weight

Mix Number	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
Extruded	-	6	3	6	5	6	3	5	3	3	5	5	5	3	5	6	6	6	6
Fit to Test	-	3	0	0	5	6	3	5	3	1	0	1	1	1	1	0	0	1	0

The specimens from mixtures number 1 did not complete the freeze-thaw tests, because the control specimens fell apart before the completion of the tests. The comments which were



written on the data sheets were copied into this report because they were of more assistance than the numerical results in the evaluation of the data. The following comments concern the specimens moulded with the lesser compactive effort.

- 1A - all specimens discarded after three cycles.
- 2A - first trial; tests were stopped after 11 cycles. Specimens were not given an intermediate soaking period. RC-3 asphalt used for topping. The one specimen fit to test had strength of 64 psi.
- 3A - tests stopped after 10 cycles; specimens were capped with type RC-3 asphalt.
- 4A - first trial: Tops of specimens coated with latex paint. Tests stopped after 7 cycles. Average strength of two specimens fit to test was 50 psi.
- 5A - cylinders were easily extruded; heaving not noticeable.
- 6A - tops disintegrated on four specimens; all samples were easily extruded.

None of the specimens just described were subject to regular temperature fluctuations during the freezing cycles; that is, some of the freezing periods were much milder than the others. Specimens of mixture 7A were the last samples to have the heater operating in the reservoir for all twelve freezing periods. Thus samples of mixtures 5A, 6A and 7A were not frozen to the full depth. In fact, it is believed that the frost line did not penetrate further than the mid-point of the specimens. Specimens marked 8A received one penetration of frost to the full depth, those of mix 9A received two such penetrations, etc. up to mixture 17A, whose samples were frozen to the full depth for every cycle.

7A - considerable evidence of ice lenses, though heaving was not noticed. The specimen tested was partially broken up by frost action and soaking.





8A - three specimens which had the toppings loosened on one side had an average strength of 68 psi. The sound specimen had a strength of 115 psi. No lenses were evident in the form of horizontal cracks on the surface of the samples, but they were easily broken into horizontal layers.

9A - all specimens were difficult to extrude. Ice lenses were evident, especially in the upper one-third of the samples.

10A - the cylinders swelled about 0.1 inch during the initial soaking period. At the end of freeze-thaw tests the upper half-inch of the specimens were very soft.

11A - five of the specimens lost their asphalt toppings and the upper part of all of the specimens broke up when soaked.

12A - the specimens separated easily into horizontal layers when handled. The tops of three specimens became very soft and yielding; the colloquial term for the appearance of the tops would be mush.

13A - samples were very difficult to extrude; the top was loose on the cylinder tested for strength. The specimens were easily broken into horizontal layers.

14A - the specimens were easy to extrude; all of the asphalt toppings floated off when the specimens were immersed. The specimens which were broken for strength determination had very rough contact faces.

15A - unable to break any cylinders because the upper surfaces disintegrated when the asphalt tops floated off.

16A - same as 15A.

All of the subsequent test cylinders received twelve cycles of freezing without heating the water in the reservoirs.

17A - no comments.

18A - the two specimens which were tested had very soft upper quarters. The asphalt tops were lost and one specimen fell completely apart when soaked.

19A - the samples lost their asphalt toppings when immersed in water. The tops were very soft with no cementing strength evident in the upper quarter inch of the specimens. Soil was lost from the tops, bottoms and cylindrical surface of the samples.

2AX - test 2A was repeated in lieu of mixture 20A. Three of the specimens heaved one-quarter inch from the bottoms of the moulds and there was some heaving from the top of the moulds as well. Frost-action was indicated in the upper two-thirds of the specimens. None of the cylinders were fit to test.

4AX - test 4A was repeated in lieu of mixture 21A. All of the specimens were difficult to extrude and all lost their tops when soaked.



Plate 3 illustrates the condition of one of the specimens after 12 cycles of freezing and thawing and 24 hours of soaking. Plate 4 shows the conditions of specimens 6A and 7A after undergoing the same test. One will note that specimens 6A and 7A still possessed their asphalt toppings whereas the top was missing from sample 19A. The soft, spongy nature of the upper portion of the specimen can be seen clearly in Plate 3. The horizontal surface cracks near the bottom of the specimen indicated where frost action had separated the soil particles. It is believed that the better conditions of specimens 6A and 7A were partly due to the fact that they were more durable than the samples of mixture 19A; but the test was not as severe for specimens 6A and 7A as it was for sample 19A. For instance, the specimens of mixtures 2AX and 4AX were not fit to test, whereas those of mixtures 2A and 4A showed strengths of over 50 psi. Thus, it was impossible to properly evaluate the relative durability of the specimens.

The following comments concern the freeze-thaw tests on specimens moulded with the higher compactive effort. The specimens received only six cycles of freezing and thawing unless otherwise stated.

1B - all samples discarded after three cycles.

2B - the specimens received 11 cycles of freezing and thawing. The average strength of the two specimens fit to test was 115 psi. Four of the cylinders were extruded intact; only one top came off, but on one other specimen the top was too soft and irregular to place in the strength testing apparatus. The tops were coated with asphalt, type RC-3.







19A41

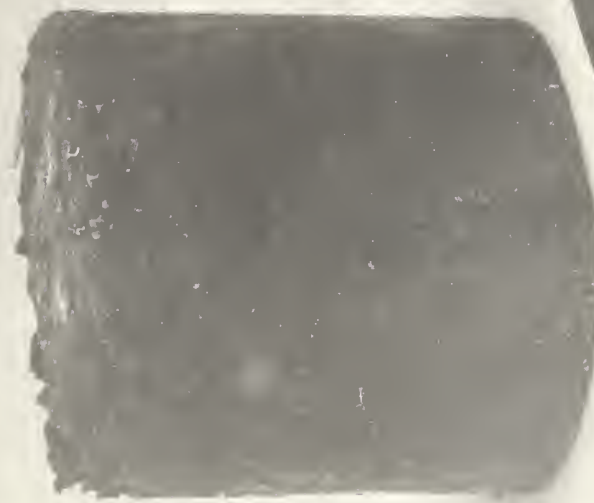
SAMPLE AFTER 12 FREEZE-THAW CYCLES  
& FINAL DAY OF SOAKING PLATE 3





6A46

7A46



SAMPLES AFTER 12 CYCLES OF FREEZE - THAW & FINAL DAY OF SOAKING  
PLATE 4



- 3B - the tops were coated with asphalt, type RC-3. The tests were stopped after the completion of ten cycles of freezing and thawing. The control specimens had crumbled within hours after being immersed in water.
- 4B - the tops of the cylinder were covered with latex paint. The tests were stopped after 7 cycles. The tops were very soft; the lower portions of some specimens was sound, while others showed signs of horizontal ice lenses.
- 5B - no comments.
- 6B - cylinders were extruded easily.
- 7B - specimens fitted tightly in the moulds.
- 8B - no comments.
- 9B - no ice lenses were apparent.
- 10B - cylinders swelled 0.1 inch when soaked before the first freezing cycle and were difficult to extrude at the end of the tests.
- 11B - specimens were difficult to extrude. The tops of the specimens disintegrated when they were immersed.

The mixtures just described were all subject to fluctuations in the freezing temperatures. Most of the cycles were milder than the one described in the procedures in section IV.3. The specimens marked 12B were the last to receive warm water from the reservoir for all six cycles of freezing and thawing. Specimens of mixture 13B were frozen to the full depth at least once; specimens of mixture 14B experienced frost penetration to the bottom face twice; and so on up to the specimens of 17B, which were frozen to the full depth for all cycles.

- 12B - five specimens were extruded intact. One sample broke into horizontal layers when handled; three others lost their asphalt toppings when soaked, and disintegrated. The one cylinder which remained intact gave a fairly good value of strength.
- 13B - the specimens not fit to test had lost their asphalt toppings and the exposed end faces were too rough to test. The remainder of the specimens appeared to be sound but were easily broken into disc-shaped layers when handled.
- 14B - all of the specimens were difficult to extrude. The one specimen, classified as fit to test, had a hole in the top caused by the soil sticking to the face of the compaction rammer. The other specimens were sound, except for the top three-eighths of an inch,



which were badly fissured but not soft.

15B - the tops of the specimens turned to mush when immersed, except for the one specimen which was fit to test.

16B - all of the cylinders were fissured and all but one were very soft on the top. All of the asphalt tops floated off during immersion. The one specimen which was not soft on the top had soil missing from the bottom face and thus could not be tested for strength.

Specimens 17B and subsequent samples were subjected to freezing and thawing cycles in which the frost line penetrated to the bottom of the cylinders for every cycle. It required approximately 9 hours to reduce the temperature of the upper surface of the specimens to below 0C. The frost then penetrated to the bottom of the sample in less than 3 hours.

17B - all of the specimens lost their asphalt caps. The top halves of the cylinders were very soft. Horizontal cracking was very prominent from the top to the bottom of the samples.

18B - the specimens were easily extruded. The tops were very spongy even before immersion in water. The specimen which was tested was not a good sample, but it was the best of the lot. The other cylinders were soft and soggy for the full depth after soaking for 24 hours.

19B - all of the specimens lost their asphalt toppings. The resultant top faces were very irregular and there was not a good sample in the lot.

2BX - none of the cylinders were fit to test.

4BX - the bottom halves of the cylinders were sound, but the tops were mush.

The Indices of Resistance to Freezing have not been calculated, because the test procedures were too inconsistent to properly compare the values. None of the freeze-thaw specimens containing accelerators were fit to test after 12 cycles of freezing and thawing and 24 hours of soaking. The specimens were soft for the full depth and the tops were very irregular. The samples containing 2 per cent calcium ligno-sulphonate were so pliable and soft that they had difficulty





maintaining their shape when subjected to no load other than their own weight.

#### V. 4 SIGNIFICANCE OF DURABILITY TEST RESULTS

The clay-lime-pozzolan mixtures stabilized in this program all failed to pass the durability test. The results from actual installations in Manitoba, which were recited in section II.5, indicated that durable mixtures could be stabilized using only 5 per cent lime and no pozzolan additive. However, it is doubtful if the field installations in Manitoba were subject to the pre-freezing soaking and the sharp rise in the water tables experienced in the post-freezing soaking by the specimens in the freeze-thaw test.

It is not likely that the field installations of clay-lime mixtures would have access to an unlimited water supply during the freezing process. The stabilized clay-lime subbase would only be placed on a clay subgrade, since the stabilization process uses the material which is most plentiful in the area. Since the flow of water through the clay subgrade would be hindered by the low permeability of the clay, the ASTM test probably approximates field conditions more closely than the modified British test. In the ASTM test, the amount of water available for freezing is limited to that contained in the specimen and in the felt pad which lies beneath the sample.

The soaking by immersion of the cylinders before testing for strength has practicable applications because a stabilized



clay subbase might be inundated in the spring by melt water which the impermeable clay subgrade could not accommodate.

Lu and others (1957) showed that the mixtures treated with dolomitic lime were the only type able to withstand deterioration by freezing and thawing, when compared to mixtures containing calcitic hydrated limes and quicklimes. It is believed that Manitoba test strips may have used dolomitic lime.

In conclusion, one might say that the durability characteristics of clay-lime-pozzolan mixtures stabilized from local materials probably range within the limits of the results shown by this test program and the results displayed in the installations in Manitoba. Thus the results of this test program hold little significance except to show that the stabilized mixtures failed to pass a test which was not representative of conditions which would exist in a highway.



## CHAPTER VI

### CONCLUSIONS AND RECOMMENDATIONS

The statement of the problem, as given in chapter I is repeated to ascertain the increase of knowledge that has been gained from the literature survey and the test program.

1. Would the 28-day strengths achieved by stabilized mixtures of local materials be sufficient to warrant their application in a highway pavement structure?

2. Were the stabilized mixtures sufficiently durable to withstand the rigorous climate of Alberta?

The answers are given with reservations because the information compiled on the subject was still insufficient to attest definitive replies. The question concerning durability remains unanswered because the published literature and the testing program failed to reveal a significant test for durability. The first question receives an affirmative reply with qualifications which will be developed in the following specific conclusions.

#### VI.1 CONCLUSION

##### The Significance of the Specified Standard Strength

Stabilized clay-lime-pozzolan mixtures could be produced that would possess sufficient strength to permit their use in a highway pavement structure provided observations of test strips over a period of years proved their durability. Davidson





and Handy (1960) suggested that immersed 28-day strengths of 500 psi would be necessary to prevent frost damage. However this was a value, printed in a handbook to cover all types of conditions and soil types; which would not necessarily be the minimum for the clays of Alberta. This report suggests that clay from Fahler, when mixed with the proper amounts of lime and pozzolan, could produce strengths that were less than 500 psi and yet might not fail due to frost damage. To substantiate the suggestion one would have to subject the stabilized mixtures to a freeze-thaw test which represents conditions existing in the Province of Alberta.

Laboratory tests cannot simulate field conditions at the present time because field observations of the moisture content, ground temperature, length and duration of freezing periods and frequency of freeze-thaw cycles are not available for this region.

#### Laboratory Conditions Can Be Duplicated in the Field

The test procedures did not seek to attain ideal conditions and therefore the strengths derived in the laboratory should be possible from a field installation. The moisture contents, at the time of mixing the test specimens, probably deviated by two or three per cent from the optimum moisture content for strength. A field installation should be able to meet a specification which allowed a range of four to six percent moisture content at the time of compaction. The initial mixing in the field would probably not be as



thorough as the hand mixing conducted in the laboratory test. However, remixing after the rotting period would help to pulverize the soil. The long, hot summer days experienced in Alberta should aid in the development of strengths, but practically no hardening would be experienced during the cool, autumn evenings. A lack of knowledge about ground temperatures makes it difficult to correlate the 28-day strengths of samples cured at 25C with the strengths in the field cured for any specified period.

#### Strength Gain With Age

The test results from this report indicated that some strength was lost initially when the specimens were immersed and commenced to swell. It was further indicated that when the swelling was completed, the initial strength was slowly recovered. Thus, the stabilized lime mixtures continued to gain in strength with the passing of time both in the immersed and partly saturated condition. A lime-stabilized installation which survived the first spring thaw should last indefinitely.

#### The Stabilized Soil Was Stronger Than the Untreated Subgrade Materials

Since the immersed unconfined strength of the untreated clay was negligible, any material possessing a higher strength could be used between the untreated clay subgrade and the overlying layers of gravel. The addition of a stabilized clay-lime-pozzolan layer would justify the reduction in thickness of the subbase, or if the stabilized soil were sufficiently strong, it might replace the



gravel subbase. Such an installation would have to be justified economically. For this purpose, the mixture which provided the greatest strength per unit capital cost would be the proper choice, even if it was not the mixture which gave the highest absolute strength.

#### Use of Accelerators

Accelerators can be used which cause early strength in stabilized lime-soil mixtures. Alkalies and basic salts have proven to be the best accelerators, but the values of strength obtained from their use are very sensitive to soil type, and to the quantity of lime-pozzolan and accelerator.

#### Effect of Carbonation

The test program was arranged to prevent carbonation of the lime during the curing period. Some authorities believed that carbonation of the lime increased the strength of the clay-lime mixtures once they were stabilized. The test results, then, may be indicative of higher possible field strengths.

#### Miscellaneous Conclusions From This Study

Durability tests. Separate durability tests were an unnecessary complement to the strength tests; and will continue to be in future studies until they are based on local field experience.





Use of six specimens. The use of six specimens was justified because the narrow limits set by the average values enabled one to criticize surely, knowing that the results correctly interpreted the behavior of the mixtures. The use of three specimens might have so blurred the results that some of the trends expounded in the previous chapter would not have been distinguished.

Rotting. Rotting of the clay-lime mixtures was necessary to ensure a thorough mixing of the constituents. When rotting was allowed, there was not a linear relationship for the density and the moulding moisture content between specimens containing lime only, and those which contained pozzolan only. A linear relationship appeared to exist, however, between specimens containing a large proportion of lime and samples with lime contents near the lime-fixation capacity of the mixtures.

Factors which affected the strength and durability. The immersed unconfined compressive strength of stabilized clay-lime-pozzolan mixtures depended upon the many factors which were outlined in section II.4 and V.2.

## VI.2 RECOMMENDATIONS ARISING FROM THIS STUDY

### Physical Comparison With Other Methods of Stabilization

The relative merits of soil-cement, soil-lime-pozzolan, soil-bitumen and trace chemicals in the stabilization of clays



occurring in Alberta should be compared. If cement and bitumen cannot be properly mixed into the soil, then lime should be added, in the amount of the lime-fixation capacity of the soil, to condition the clay for cement or bitumen stabilization.

#### Economical Comparison of Stabilizing Agents

Following the preceding collation, an economic appraisal should compare the various methods of stabilization. If lime stabilization should prove to be economically feasible, then the remainder of these recommendations should be considered, but the final recommendation should be investigated regardless of the outcome of the economic survey. The first few items are suggestions for short test programs to bind together the loose ends of this investigation.

#### Check on Activity of Lime and Pozzolan

A study of the strength of the lime and pozzolan mortars for various lime-pozzolan ratios would be helpful. The results could be compared to similar mixtures of reagent grade calcium hydroxide and the pozzolan. Thus the efficiency of the lime could be established.

Perhaps other pozzolans could be tried even though the sources have not as yet been developed for commercial purposes. If lime stabilization proved to be a successful replacement for the present granular subbase, the large market would soon provide commercial development of any favorable source of supply.



### Optimum Moisture Content For Strength

The optimum moisture content for strength of the mixtures which had lime-pozzolan ratios greater than one should be determined. These moisture contents would likely be different from the optimum moisture content for density.

### Strength Tests on Other Mixtures

It was supposed that high values of unconfined compressive strengths would result from stabilized batches containing 15 per cent each of lime and pozzolan by weight of the clay.

Mixtures 20 and 21 should also be blended and tested for strength, for although relatively weak, they may have economical applications.

### Rotting Time

Since the rotting time affected the density and moulding moisture content, it presumably made an impression upon the strength also. The effect of variations in the rotting time upon the optimum moisture contents for strength should be investigated.

### Effect of High Temperature Curing

Several investigations have been conducted on mixtures cured at 50C. A correlation between the 7-day strengths at 50C and the 28-day strengths at 25C would be helpful in shortening the testing programs.

Since applied heat seemed to benefit strength, perhaps





chemical reactions which were exothermic should be encouraged in the mixtures. The heat evolved from the slaking of the lime may be one of the reasons why soils stabilized with quicklime often reached higher strengths than if hydrated lime was used. The same thought could be kept in mind when one chooses an accelerator for a trial mix.

#### Curing at Low Temperatures

A knowledge of the strengths of specimens cured at temperatures between 0C and 25C is vital because the cool nights, which are prevalent in the Province of Alberta during the warm months, tend to lower the mean daily temperature. Except for the work of Dumbleton and Ross (1960), very little research has been reported at these temperatures.

#### Length-Diameter Ratio

A method of compacting specimens with a length to diameter ratio of two should be evolved. The present size of specimen is valuable because it can be compacted in one lift. However, the strengths from the present investigation are subject to a correction because the friction at the end faces tended to increase the compressive stress necessary to break the specimen.

#### Dolomitic Lime

A series of 28-day strength tests similar to those conducted in this test program should be completed using a commercial dolomitic lime. The higher costs involved may erase any of the benefits gained in strength, but if stabilization with dolomitic lime were accepted as a highway subbase material, the



prospective market would probably cause a search for dolomitic limestone properties in Alberta that could be commercially developed.

### Carbonation

The rate of carbonation of the lime-pozzolan mixtures should be analyzed. The test procedure used by the Alberta Soil Survey to discover the quantity of carbonate in a sample is relatively simple, rapid and economical. Although one cannot discover the strength due to carbonation by direct means, it could be found by allowing one part of a batch to cure in a carbon dioxide atmosphere while preventing the other portion from absorbing the gas. The difference in strengths would be attributed to carbonation. The concentration of carbon dioxide could be varied to discover the resultant effect on the unconfined compressive strength. Tests would have to determine the gain in carbonate content in the specimens being cured in an atmosphere supposedly free of carbon dioxide; such as those in the 2 pound polyethelene bags used in this test program.

### Field Temperature and Moisture Conditions

A laboratory durability test must be designed which will successfully predict the results of field trials. Before any further work should be attempted on a laboratory test, however, an investigation of several years duration should record the moisture conditions in and under the pavement at all times of the year, and if possible for various times in the day as well.

This would be expensive research, but the information



cannot be gained from sources outside the province.

### Test Strips

An alternate method to the use of a laboratory durability test would entail the construction of test strips in the field. If these test strips could be combined with the instrumentation mentioned in the preceding recommendation, then it is quite certain a satisfactory laboratory test could be devised to duplicate the results of the field trials. If the instrumentation was not carried out, then the design of lime-stabilized mixtures would revert to design by experience and extrapolation similar to the method used for subgrades and pavements prior to the origination of the proctor and CBR tests.

### Activated Bitumen and Lime

At the Fifth International Conference on Soil Mechanics and Foundation Engineering, Litvinoff and others (1961) described the construction of highways in the Moscow Region using clay soils successfully stabilized with lime and activated bitumen mixtures. These mixtures were obviously durable, and since bitumen is in plentiful supply in Alberta, a local investigation to prove the Russian method is certainly warranted.





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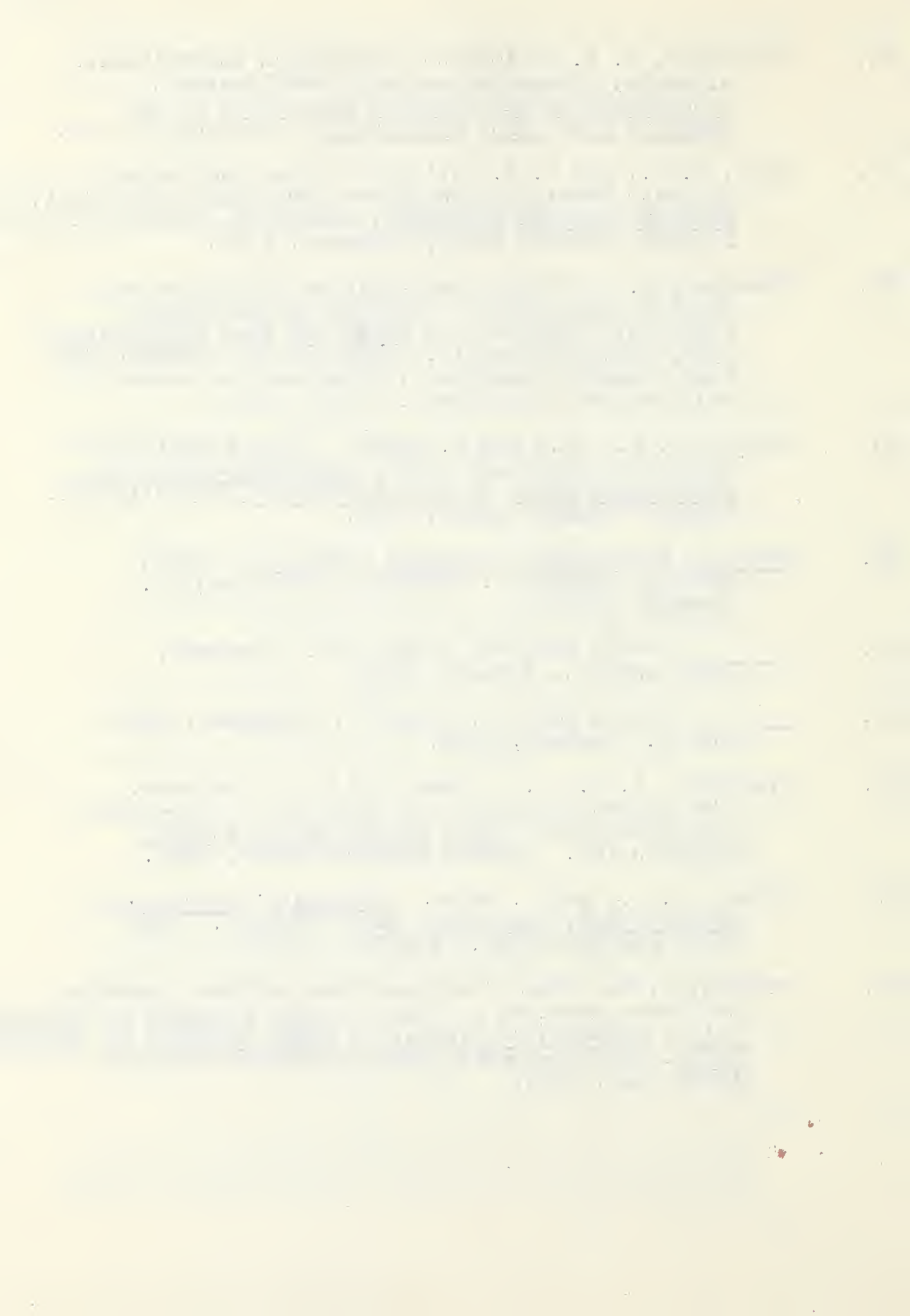
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## APPENDIX A

### CLASSIFICATION TEST PROCEDURES

#### I. SPECIFIC GRAVITY

The following procedure was used to determine the specific gravity of the clay and the pozzolan.

1. A 500 milliliter flask was calibrated to determine the weight of the flask filled with water at temperatures between 20C and 30C.
2. The clean, dry flask was weighed to the nearest 0.01 grams.
3. Approximately 50 grams of soil was placed in the flask and the weight of the air-dry soil in the glass was determined to the nearest 0.02 grams. A second sample of the same soil was used to determine the moisture content of the air-dried soil. The oven-dry weight of the soil was calculated to the nearest 0.03 grams.
4. Approximately 250 milliliters of de-aired, distilled water was added slowly to the flask and the soil was mixed into the water by gently shaking the flask.
5. The mixture was allowed to soak overnight and then was de-aired for twenty minutes under an absolute pressure of 0.05 Kgms/cm<sup>2</sup>.
6. The vacuum was released at the completion of the de-airing procedure, and water was added until the level reached the neck of the bottle. Complete de-airing was checked by



reapplying the vacuum and watching for a significant rise in the water level. If the rise of the water level in the neck of the bottle was less than one-half inch, the de-airing was considered complete.

7. The flask was filled to the calibration mark with water, and weighed to the nearest 0.01 grams.

8. The temperatures of the mixture was taken immediately after recording the weight of the flask filled with water and soil.

9. The weight of the flask filled with water was calculated from a calibration curve using the temperature obtained in 8.

10. Having determined the weight of soil,  $W_s$ ; the weight of the flask filled with water and soil,  $W_{b+w+s}$ ; the weight of the flask filled with water,  $W_{b+w}$ ; the specific gravity was calculated from the equation:

$$G_s = \frac{W_s}{W_{b+w} + W_s - W_{b+w+s}}$$

## II. THE ATTERBERG LIMITS

### Liquid Limit

The procedure outlined in ASTM D423-54T, was used to determine the liquid limit except the soil was soaked overnight at approximately the liquid limit before the test was conducted.



### Plastic Limit

The procedure outlined in ASTM D 424 - 54T was followed to determine the plastic limit except the soil was soaked overnight at approximately the plastic limit before the test was conducted.

### Shrinkage Limit

The procedure outlined in ASTM D 427-39 was followed in the determination of the shrinkage limit.

## III. GRAIN SIZE ANALYSIS

A hydrometer analysis was used to determine the grain size distribution of the clay. The following detailed procedure was used in the test:

1. A moist specimen of soil, representing approximately 50 grams dry weight, was mixed with distilled water to form a smooth thin paste.
2. 10 cc's of 6%, by weight, solution of sodium hexametaphosphate was added to the paste as a dispersing agent.
3. The mixture was washed into a dispersion cup, and was stirred mechanically by a milk shake mixer for ten minutes.
4. The specimen, after mixing, was washed into a graduated cylinder and distilled water was added to bring the level of the solution to the 1000 cc mark.





5. The soil and water were mixed in the graduate by placing the palm of the hand over the open end and turning the graduate upside down. The graduate was vigorously shaken to loosen any soil that was stuck to the bottom of the graduate.

6. The mixing continued until the graduate had been inverted a total of thirty times. The graduate was then placed on a table and a timer was started.

7. Hydrometer readings were taken at total elapsed times of  $\frac{1}{4}$ ,  $\frac{1}{2}$ , 1 and 2 minutes without removing the hydrometer.

8. The suspension was then remixed and the test was restarted with the first reading taken at 2 minutes; followed by readings at 4, 8, 15, 20 and 60 minutes and at approximately 2, 4, 8, 12 and 24 hours. The hydrometer was removed from the suspension after each reading.

10. After the final reading, the suspension was washed into an evaporating dish and the weight of the soil was determined by oven drying the specimen.

11. The hydrometer had been previously calibrated and a meniscus correction was added to each reading.

12. Corrections for the volume of the hydrometer were made for the readings that were two minutes and less.

13. The corrected readings were then used to find limiting diameters for the various elapsed times by means of a nomograph, which had been previously calibrated for the hydrometer.

14. Meniscus, temperature, and density corrections were



applied to the original readings and the resultant corrected readings were entered into the following equation:

$$W\% = \frac{100}{W_s} \cdot \frac{G}{G - 1} \cdot R_h$$

where G was the specific gravity of the soil

$W_s$  was the weight of oven-dry soil in grams

$R_h$  was the corrected hydrometer reading

and  $W\%$  was the percentage of the total dry weight of soil finer than the limiting diameter, calculated in 13, for each elapsed time.

15. The results were plotted on a graph of log of diameter versus the per cent finer than each diameter.

16. The percentages of sand, silt and clay sizes were read directly from the graph.



## APPENDIX B

### MOISTURE-DENSITY RELATIONSHIPS

#### I. STANDARD PROCTOR TEST RESULTS

The standard proctor tests were run according to ASTM D 698-57T, method A, except that the soil was allowed to soak for 24 hours before being compacted. A total of three standard proctor tests were run on the soil and the results of these tests are plotted on Figure B1.

#### II. MODIFIED AASHO TEST RESULTS

The modified AASHO test was conducted in accordance with ASTM proposed method of finding the Moisture-Density Relations of Soils Using a 10-lb. Rammer and an 18-in. Drop, Method A, with the exception that the soil was allowed to soak for 24 hours between the times of mixing and of compaction. Two tests were run on the soil and the results are plotted on Figure B1.

#### III. MOISTURE-DENSITY TESTS USING A 5-LB. HAMMER DROPPED 12-INS.

A TOTAL OF 5 TIMES ON EACH END OF A 2-IN. SPECIMEN

#### Determination of Weight of Soil

The optimum moisture content was assumed to be the same as was found in the standard proctor test. The first trial specimen used 160 grams of air-dried soil. The water and soil





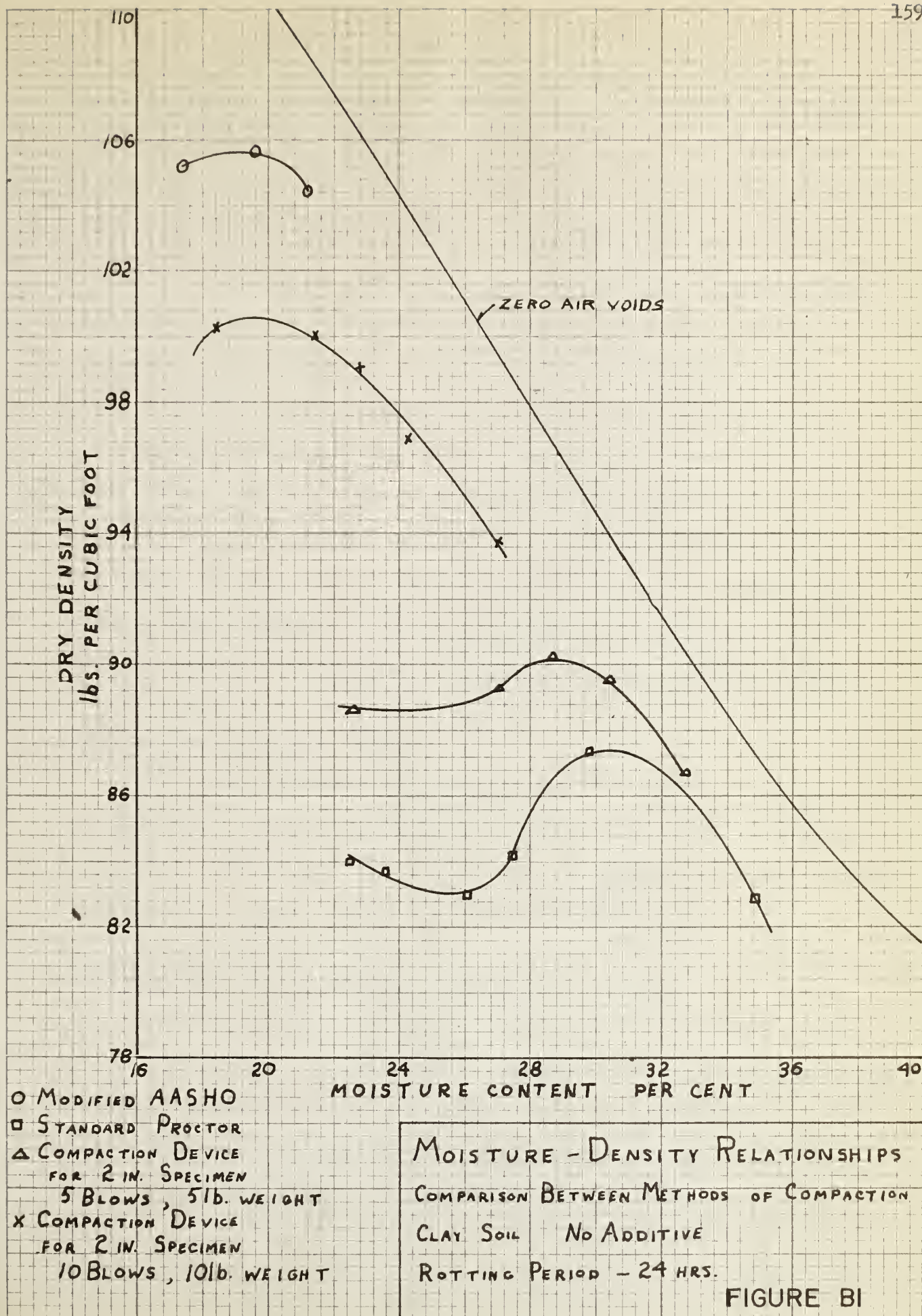


FIGURE BI



were thoroughly mixed and one specimen was compacted, extruded from the mould, and the height was measured. The weights of the water and soil were then varied in the same proportions that the height varied from 2 inches. Successive trials were run until the height measured 2.00 inches  $\pm$  0.01 inches. Five specimens, each of different moisture content, were then compacted. The range of moisture contents varied from 4% above to 4% below the optimum moisture content obtained from the standard proctor test. The optimum density and the moisture content for the 2-inch specimen were determined by plotting the dry unit weight against the moisture content. If the height at the optimum density differed by more than 0.010 inches from 2 inches, then the weights of the soil and the water were varied to produce specimens of the correct height.

#### Evaluation of Rotting Time.

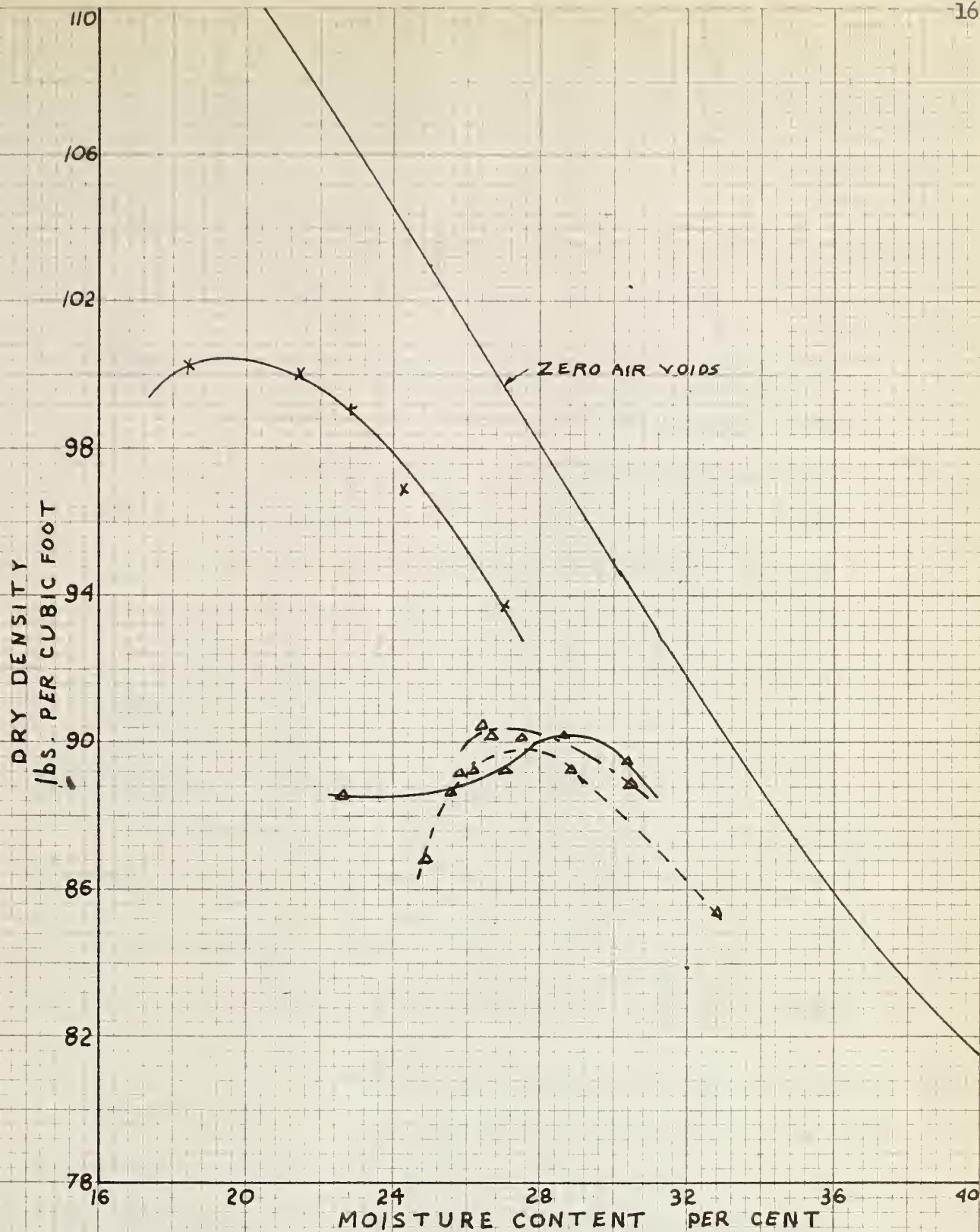
Moisture-Density curves were run on the soil, at various times after completion of mixing. The times between mixing and compaction that were tried included 0, 3 and 24 hours. The results of these tests are shown on Figure B2.

#### Determination Of The Weight of Lime-Soil Mixture

The procedures followed to determine the weight of dry soil at the optimum density were followed when lime in the amount of 20% of the dry weight of the soil was mixed with the air-dry soil. However, the assumed optimum moisture content was 6% higher than the optimum moisture content for the plain soil.







COMPACTION DEVICE  
FOR 2 IN. SPECIMEN

Δ 5 BLOWS, 5 lb. WEIGHT

x 10 BLOWS, 10 lb. WEIGHT

— 24 HOUR ROTTING PERIOD

- - - 3 HOUR ROTTING PERIOD

- - - NO ROTTING PERIOD

MOISTURE - DENSITY RELATIONSHIPS

COMPARISON OF ROTTING TIME

CLAY SOIL NO ADDITIVE

FIGURE B2





The results of the tests are plotted on Figure B3.

#### Rotting Time of Lime-Soil Mixtures

Moisture density curves were run for rotting periods of 0, 3 and 24 hours. The results were plotted on Figure B3.

#### Determination Of The Weight Of Pozzolan-Soil Mixtures

The procedures followed to determine the weight of dry soil at the optimum density were followed when pozzolan was added in the amount of 20% of the dry weight of the air-dried soil. In this case, however, the assumed optimum moisture content was 2% lower than the optimum for the plain soil. The results of the moisture density test are plotted on Figure B4.

#### Rotting Time of Soil-Pozzolan Mixtures

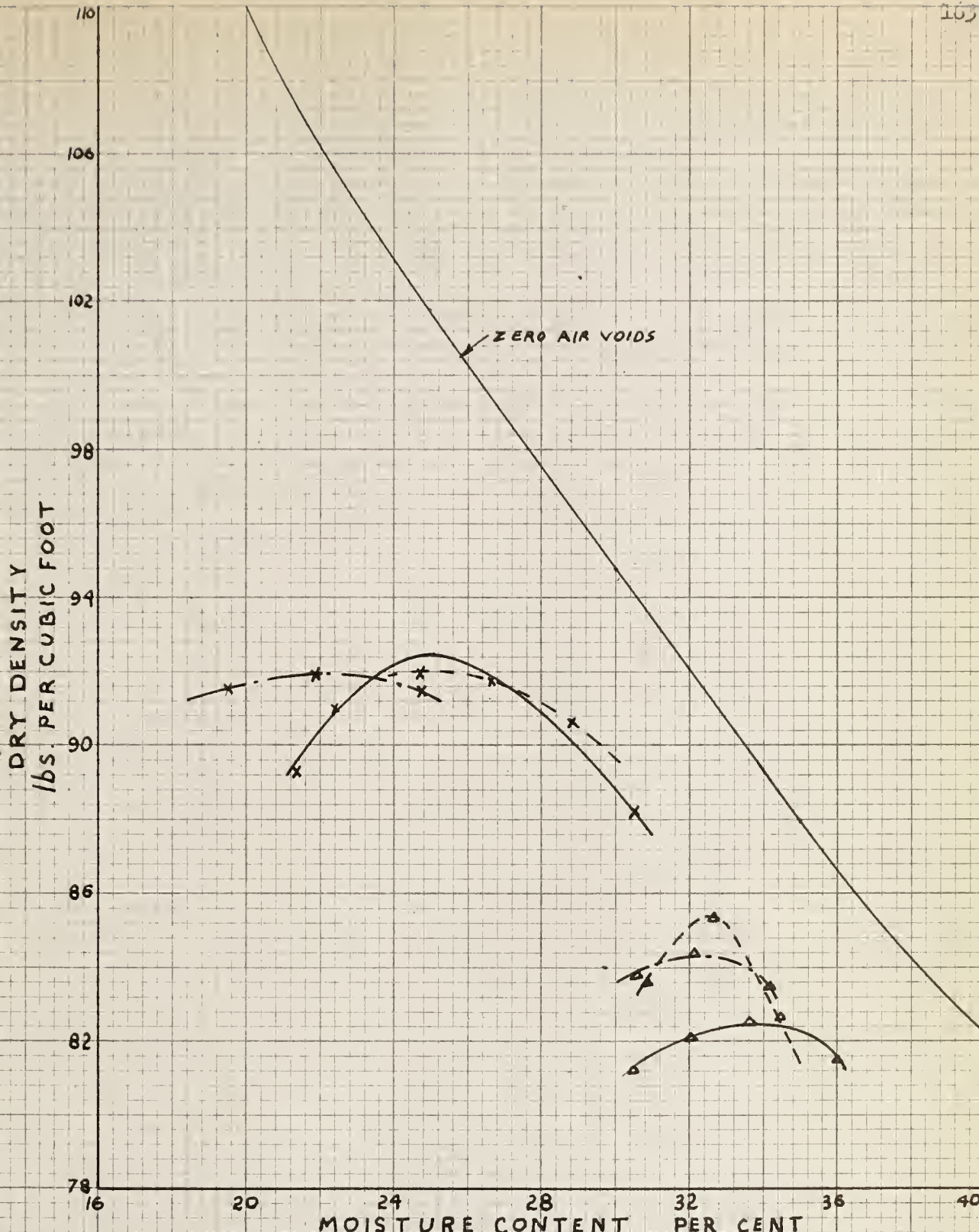
Moisture density curves were run for rotting times of 0, 3 and 24 hours. The results were plotted on Figure B4.

#### IV. MOISTURE-DENSITY TESTS USING A 10-LB. HAMMER DROPPED 12-INS.

A TOTAL OF 10 TIMES ON EACH END OF A 2-IN. SPECIMEN

The test procedures were the same as were followed for the lesser density; except the assumed optimum for the lime-soil mixture was only 4% higher than the optimum for the plain soil. The results are plotted on Figures B2, B3 and B4 for the respective mixtures of plain soil, soil-lime, and soil-pozzolan. Rotting times other than 24 hours were not tried for the plain soil.





COMPACTION DEVICE  
FOR 2 IN. SPECIMEN

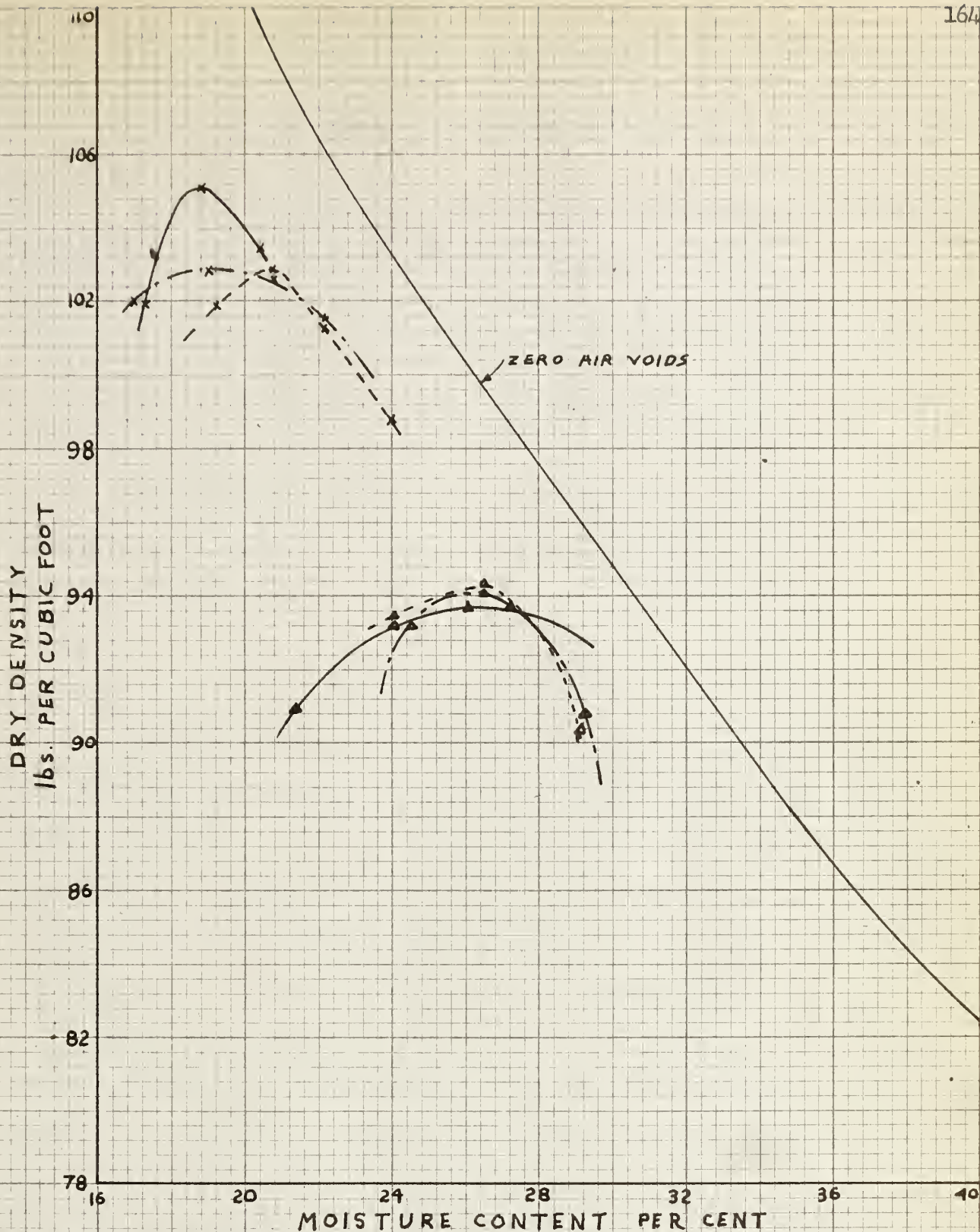
- Δ 5 BLOWS, 5 lb. WEIGHT
- X 10 BLOWS, 10 lb. WEIGHT
- 24 HOUR ROTTING PERIOD
- - - 3 HOUR ROTTING PERIOD
- · · NO ROTTING PERIOD

MOISTURE - DENSITY RELATIONSHIPS  
COMPARISON OF ROTTING TIME  
CLAY SOIL PLUS 20% LIME

FIGURE B3







COMPACTION DEVICE  
FOR 2 IN. SPECIMEN

△ 5 BLOWS, 5 lb. WEIGHT

x 10 BLOWS, 10 lb. WEIGHT

— 24 HOUR ROTTING PERIOD

- - - 3 HOUR ROTTING PERIOD

- - - NO ROTTING PERIOD

MOISTURE - DENSITY RELATIONSHIPS  
COMPARISON OF ROTTING TIME  
CLAY SOIL PLUS 20% POZZOLAN

FIGURE B4





## V. VARIATION IN THE NUMBER OF BLOWS

A moisture-density curve was run using the 5-lb. hammer, dropping a distance of 12 inches, a total of 4 times on each end of the 2-inch specimen after the soil had rotted for 24 hours. A second curve was run using the 10-lb. hammer dropped a distance of 12 inches, a total of 12 times on each end of the 2-inch specimen after the soil had rotted a period of 24 hours.

The results of the two tests are plotted together with the standard tests, and the results of the standard proctor and modified AASHO tests, on Figure B5.

## VI. COMPACTION PROCEDURE

The apparatus is portrayed in Plate B1. The mould was set over the stationary bottom piston and rested on the steel wedge. The top of the mould was set in place and the soil was poured from the bag into the mould. The rammer was set on top of the loose soil and the sample was given one blow. The wedge was then removed and the weight of the mould was held by the friction between the soil and the wall of the mould. The sample was given four additional blows, then the top of the mould was removed, the mould was turned end for end and the specimen was given five more blows.



MODIFIED AASHO O

DRY DENSITY  
lbs. PER CUBIC FOOT

102

106

110

98

94

90

86

82

78

90% MODIFIED

12 BLOWS PER FACE  
95% MODIFIED  
10 BLOWS PER FACE X

5 BLOWS PER FACE

4 BLOWS PER FACE

□ STANDARD PROCTOR

COMPACTIVE EFFORT  $\times 10^3$  FT. lbs. PER CUBIC FT.

O MODIFIED AASHO

□ STANDARD PROCTOR

COMPACTION DEVICE

FOR 2 IN. SPECIMEN

• Δ 5 lb. WEIGHT

- - X 10 lb. WEIGHT

COMPARISON OF COMPACTION TESTS

DRY DENSITY VERSUS COMPACTIVE EFFORT

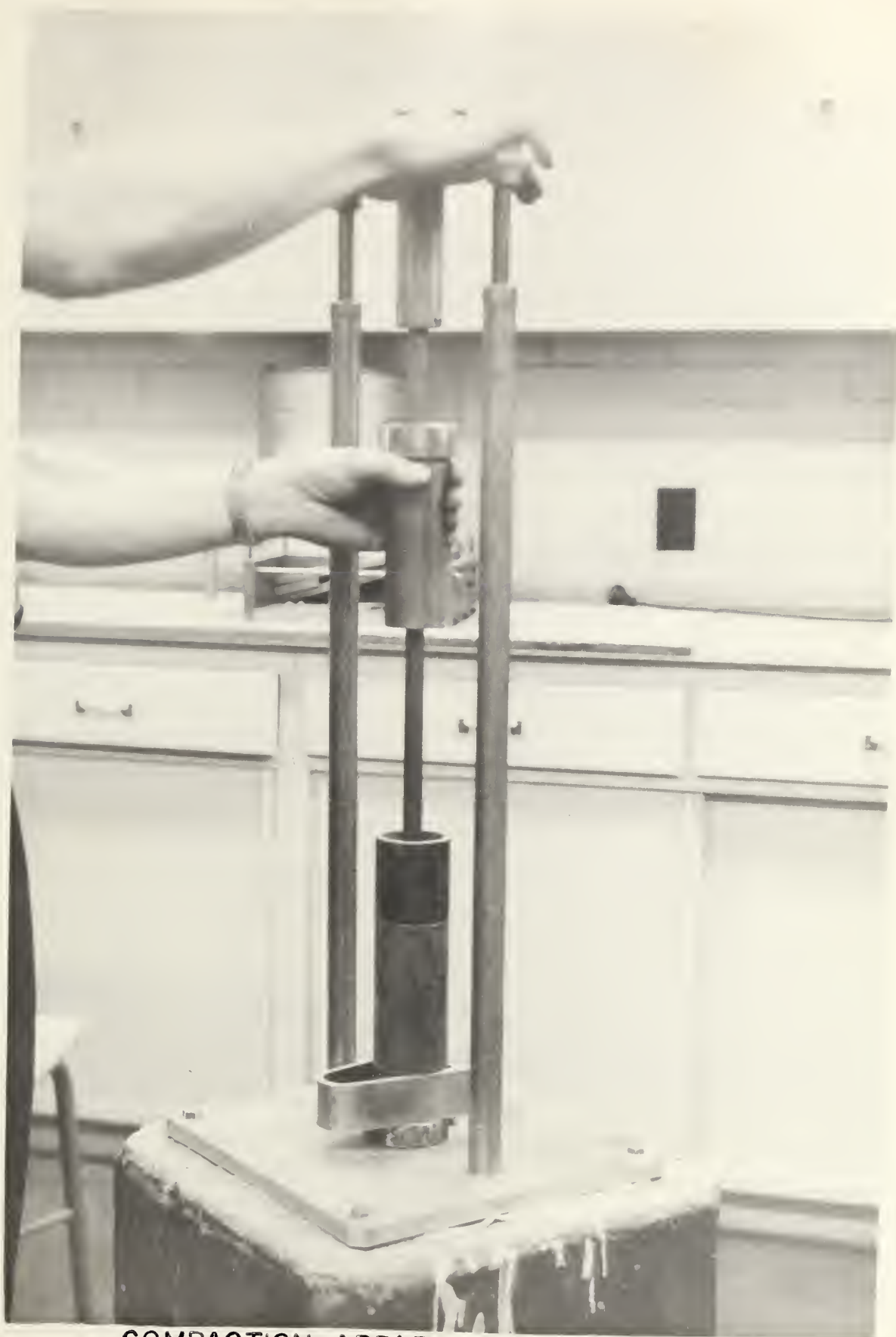
CLAY SOIL NO ADDITIVE

ROTTING PERIOD - 24 HRS.

FIGURE B5







COMPACTION APPARATUS - 5 LB. WEIGHT  
PLATE BI





## VII. INITIAL MEASUREMENTS AND CALCULATION OF THE COMPACTED DRY DENSITY

### Measurement of Height

The height of each cylinder was measured by means of an Ames dial, which was attached to a retort stand as shown in Plate B2. The Ames dial was zeroed by using a short steel rod as a standard. A sheet of polyethelene was placed under the specimens for all measurements of height and weight as an aid to cleanliness.

Each specimen was measured for height in the center of the face, then the specimen was turned upside down, and measured again. The average of the two measurements was recorded. If the two measurements differed by more than 0.010 inches the cylinder was re-examined for defects and the ends were retrimmed. A total of five height measurements taken at various sectors of the face in both the upright and reverse positions did not improve the accuracy of the average measurements.

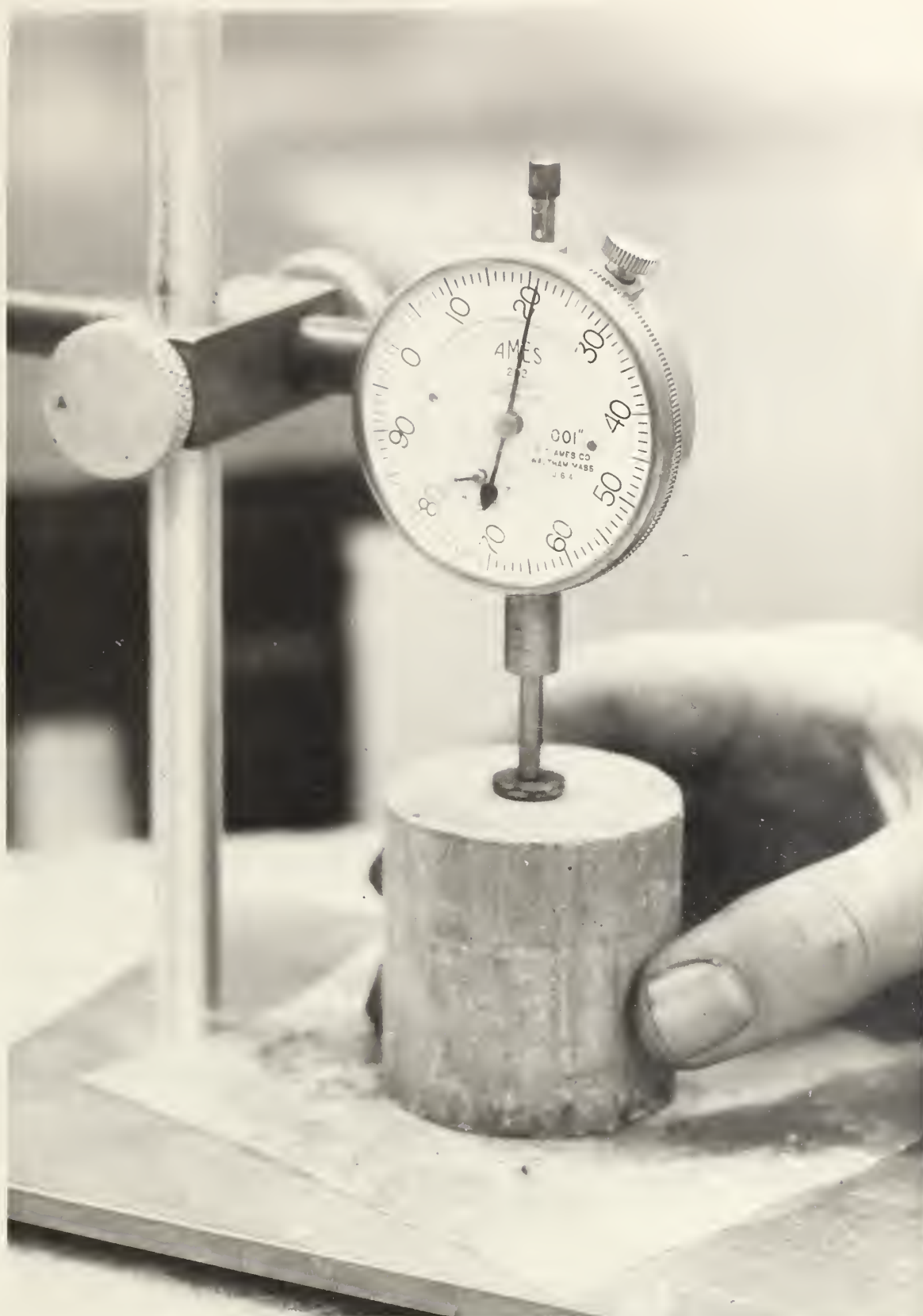
### Measurement of Diameter

The diameters of the specimens were assumed to be the same diameter as the moulds.

### Calculation of the Wet Densities

The initial wet density was calculated by means of





MEASURING HEIGHT OF SPECIMEN  
PLATE B2



the equation:

$$\gamma = C \times \frac{W}{H}$$

where W was the weight of the specimen in grams,

H was the height of the specimen in inches,

$\gamma$  was the wet density in pounds per cubic foot,

and C was a constant dependent upon the diameter of the mould. Values for C and the diameter of the moulds are given in tabular form in the following tables.

<u>Mould Number</u>	<u>Diameter in inches</u>	<u>C</u>
1	1.998	1.2150
2	1.999	1.2136
3	2.000	1.2126
4	2.004	1.2084

#### Calculation of Dry Density

Since only one moisture content was obtained for each mixture, the average wet density was calculated from the results of the 18 specimens, and the dry density was calculated using the average wet density and the moisture content. Portions of the moisture content sample were taken at the beginning and at the end of the period, during which the loose soil mixture was weighed into individual lots and sacked.





## APPENDIX C

### CALIBRATION OF PROVING RING

The proving ring used in the tests was a 2000 lb. ring from Soiltest Inc., Chicago, U. S. A. The mean result of the calibrations is plotted on Figure C1. To use the calibration chart, one multiplies the approximate value of the load per division by the number of divisions covered by the stress dial and obtains the applied load.



LOAD PER DIVISION IN lbs

No. of DIVISIONS

- BALDWIN TESTING MACHINE
- TINIUS - OLSEN TESTING APPARATUS

CALIBRATION CURVE  
2000 POUND PROVING RING  
No. 3768





## APPENDIX D

### FREEZE-THAW APPARATUS

The apparatus consisted of a wooden box, which when filled with water provided a continuous supply of water to the specimens. The specimens were held in moulds made from sections of 2-inch diameter, 50 psi, polyethelene tubing as manufactured by Alberta Polytubes Limited, Edmonton, Alberta. The moulds were held in holes drilled in the plywood cover of the box. The tubing was machined on both ends to the exact length of 2.08 inches and was fastened in place by Ev-A-Grip, an adhesive manufactured by the Wendar Adhesive Corporation, Edmonton. Cover plates of acrylic plastic, one quarter inch thick were machined to fit the bottom of the moulds. A concentric groove, 0.08 inches deep and 0.15 inches in width was cut into the face of the plates so that the plates could be slipped onto the ends of the moulds. The plates were fastened to the moulds by Reanite Adhesive or by 3M Adhesive No. EC 1368. This bond had to be strong enough to support the weight of the specimen, but weak enough to permit removal of the plates at the times set for extrusion of the specimens. Seven  $\frac{1}{2}$ -inch holes were drilled in each plate to allow passage of the water into the specimens.

The boxes were constructed of  $\frac{3}{4}$ -inch fir plywood. The bottoms were dapped into the sides and ends, and the ends were let into the sides of the boxes. The assembly was



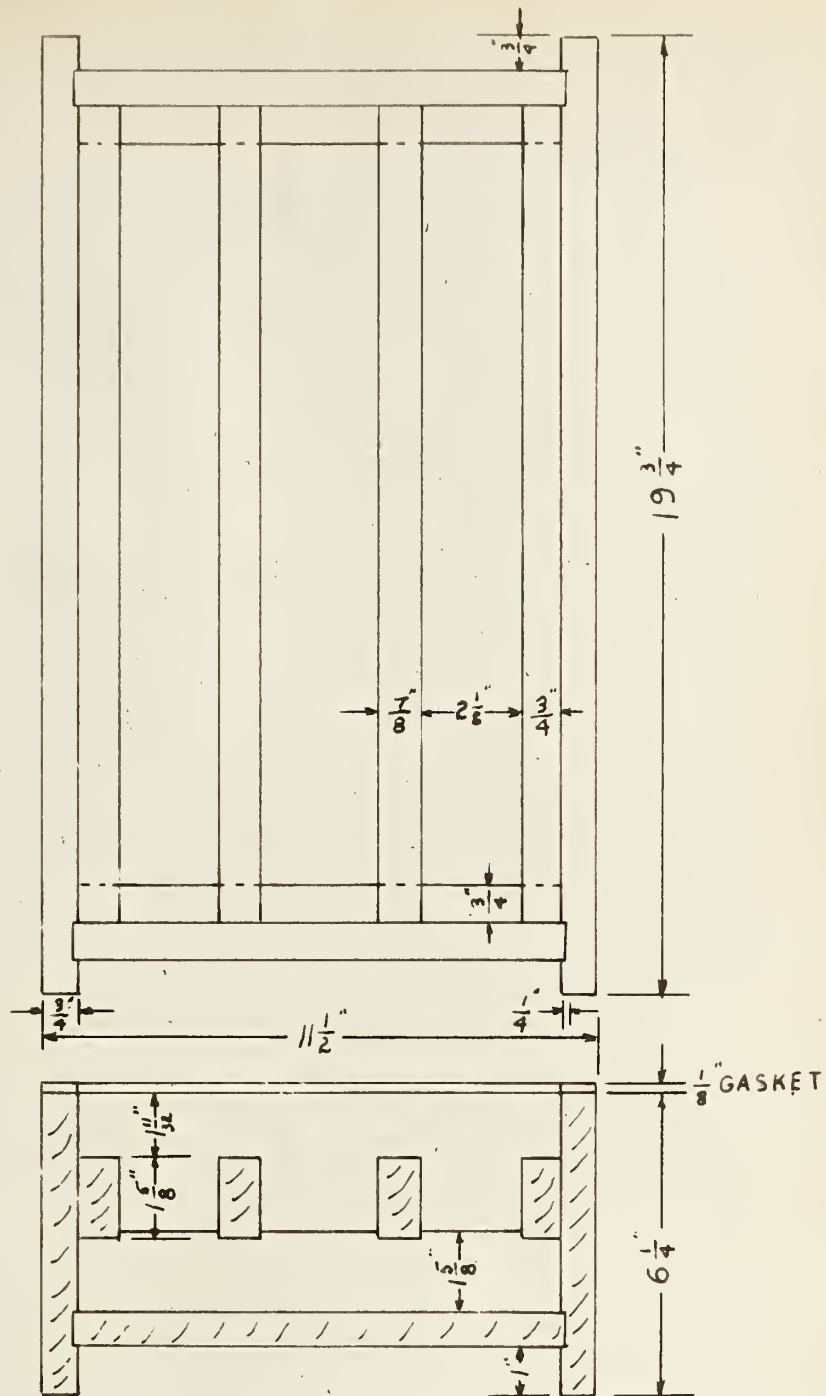


held together with Ev-A-Grip adhesive only, as no nails were used in their construction. The boxes were painted with 2 coats of a clear wood sealer in an attempt to waterproof them. Only 2 out of the 5 boxes remained watertight for the entire test period of eight weeks, although none of the boxes made to this specification developed a serious leak in that time.

Solid pieces of wood were added under the plates to prevent the plates from being pushed off into the water by the action of the frost. Their performance was good, but with a highly susceptible soil the specimen would be extruded from the bottom of the mould by the frost and the whole lid would be raised.

The plans for the boxes are shown on Drawings D1 and D2.





RESEARCH COUNCIL OF ALBERTA

FREEZE THAW APPARATUS

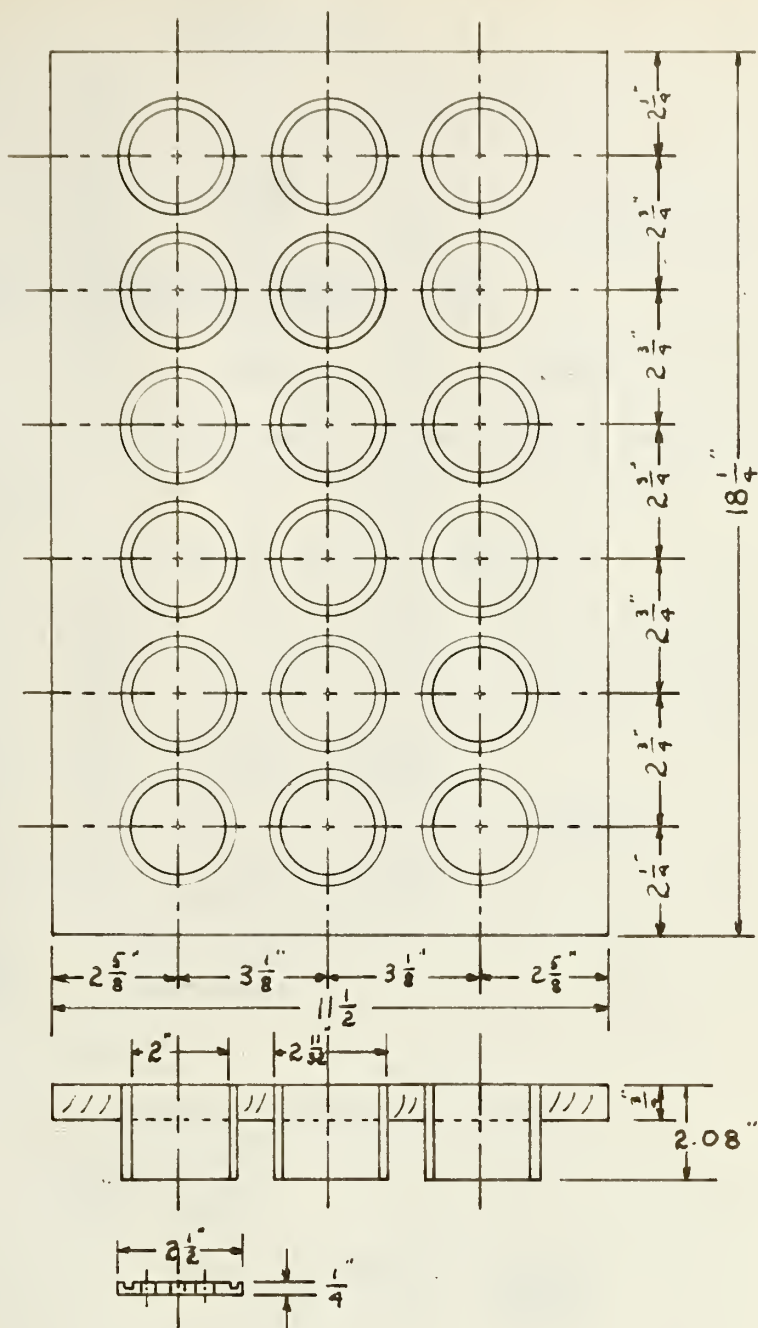
RESERVOIR

SCALE: 3"=1FT. DATE: AUG 29/61

am.

DWG. DI





RESEARCH COUNCIL OF ALBERTA

FREEZE THAW APPARATUS

RESERVOIR LID

SCALE: 3"=1FT. DATE: AUG 29/61

am.

DWG. D2





## APPENDIX E

### DETERMINATION OF THE VOIDS-CEMENT RATIO<sup>1</sup>

The voids-cement ratio of a mix is defined as:

$$\frac{\text{Absolute volume of air and water}}{\text{Absolute volume of cement}}$$

When all air is expelled from the mixture it is clear that the expression corresponds to the better known water/cement ratio (by volume).

Let  $V$  = total volume of soil cement,

$w_s, w_c$  = weights of soil and cement respectively,

$\gamma_s, \gamma_c$  = unit weights of soil and cement respectively,

$\gamma_w$  = unit weight of water,

$p$  = proportion of cement by weight.

Then absolute volume of solids =

$$\frac{w_c}{\gamma_c} + \frac{w_s}{\gamma_s} = w_s \left( \frac{p}{\gamma_c} + \frac{1}{\gamma_s} \right)$$

since  $w_c = p w_s$

Therefore, absolute volume of voids

$$= V - w_s \left( \frac{p}{\gamma_c} + \frac{1}{\gamma_s} \right) \quad \dots \dots \dots (1)$$

$$\text{and absolute volume of cement} = \frac{p w_s}{\gamma_c} \quad \dots \dots \dots (2)$$

From equations 1 and 2,

$$\text{Voids/cement ratio} = \frac{V \gamma_c}{p w_s} - \frac{\gamma_c}{p} \left( \frac{1}{\gamma_s} + \frac{p}{\gamma_c} \right) \quad \dots \dots (3)$$

---

<sup>1</sup>From appendix of article by Larnach (1960). See Bibliography at end of text.

# THEORY OF THE EARTH

THEORY OF THE EARTH

## CHAPTER I

THEORY OF THE EARTH

THEORY OF THE EARTH

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$$\text{Put } W = w_s + w_c = w_s (1 + p),$$

$$\text{then } w_s = \frac{W}{1 + p}$$

Substituting in equation 3,

$$\text{Voids/cement ratio} = \frac{V}{W} \gamma_c \left( \frac{1+p}{p} \right) - \frac{\gamma_c}{p} \left( \frac{1}{\gamma_s} + \frac{p}{\gamma_c} \right) \quad (4)$$

Now, noting that  $\frac{W}{V} = \gamma_D$ , the dry density of the mixture, rearrangement of equation 4 gives

$$\text{Voids/cement ratio} = \frac{G_c}{\gamma_D} \gamma_w \left( \frac{1+p}{p} \right) - \left( \frac{G_c}{p G_s} + 1 \right) \quad \dots (5)$$

where  $G_c$ ,  $G_s$  are the specific gravities of cement and soil respectively.

In this particular study, the lime and pozzolan are taken together as a substitute for the cement. Since the specific gravity of the pozzolan was 2.73 and of the lime was 2.76 a composite specific gravity of 2.75 was used for the additive.  $G_s = 2.78$  and  $\gamma_w = 62.43$  lbs. per cu. ft. Therefore for our purposes, the void/cement ratio was calculated from the following equation:

$$\frac{V}{C} \text{ ratio} = \frac{171.7}{\gamma_d} \left( \frac{1+p}{p} \right) - \left[ \frac{0.989}{p} + 1 \right]$$

<u>Values of p</u>	<u>Values of void/cement ratio</u>
0.05	$\frac{3605}{\gamma_d} - 20.78$
0.10	$\frac{1889}{\gamma_d} - 10.89$
0.15	$\frac{1316}{\gamma_d} - 7.59$
0.20	$\frac{1030}{\gamma_d} - 5.94$



## APPENDIX F

### SAMPLE DATA SHEETS









RESEARCH COUNCIL OF ALBERTA  
DIVISION OF HIGHWAY RESEARCH  
SOIL STABILIZATION DATA SHEET

PROJECT \* SOIL - LIME - POZZOLAN  
SOIL DESCRIPTION Clay - Fahler - CH  
ACCELERATOR DESCRIPTION Nil

TECHNICIAN gw TEST NO. 5B

DATE MIXED June 19/61 DATE COMPACTED June 20/61 DATE END OF CURING July 18/61  
28 DAY ✓ FREEZE-THAW CONTROL DATE TESTED July 19/61  
% ADDITIVE 20 LIME-POZZOLAN RATIO 5:5 ACCELERATOR

Sample number	21	22	23	24	25	26
Compacted height ins.	2.057	2.058	2.054	2.050	2.055	2.054
Compacted weight gms.	195.78	196.03	195.92	195.68	196.00	195.55
Compacted wet density pcf.	115.5	115.6	115.8	115.8	115.8	115.5
Compacted volume c.i.	6.456	6.459	6.446	6.434	6.449	6.446
Total freeze-thaw cycles						
Diam. before compression	2.021	2.018	2.015	2.018	2.019	2.017
Area before compression	3.208	3.198	3.189	3.198	3.202	3.195
Length before compression	2.079	2.082	2.076	2.075	2.078	2.077
Volume before compression	6.669	6.659	6.620	6.637	6.653	6.636
Strain dial at failure	.448	.442	.444	.450	.442	.444
Strain dial init. reading	.375	.373	.376	.377	.375	.382
Strain INS/inch	.037	.035	.034	.037	.034	.031
Corrected area ins.	3.33	3.31	3.30	3.32	3.31	3.30
Load dial divisions	247.	336.	236.	228.	234.	219.
Pounds per division	3.96	3.97	3.97	3.97	3.97	4.00
Stress p.s.i.	294.	283.	284.	273.	281.	264.
Increase in volume %	3.30	3.10	2.70	3.16	3.10	2.95
Wt. before compression gms.	205.40	205.58	204.84	205.38	205.72	204.90
Dry density at failure pcf.	91.25	91.32	91.81	91.32	91.54	91.41

MOISTURE CONTENT

No. container						
Wt. sample wet + tare gms.	115.95	163.28	122.18	123.62	122.23	150.68
Wt. sample dry + tare gms.	99.07	136.05	103.95	106.97	101.50	126.00
Weight of water gms.	16.88	27.23	18.23	16.65	20.73	24.68
Tare container gms.	40.09	41.20	39.61	49.62	29.38	39.93
Weight of dry soil gms.	58.98	94.85	64.34	57.34	72.12	86.07
Moisture content %	28.61	28.71	28.33	29.04	28.74	28.67

AVERAGE STRESS 280 psi INITIAL MOISTURE CONTENT 23.6 %  
CORRECTED STRESS  psi AVERAGE COMPACTED DRY DENSITY 93.5 pcf  
AVERAGE INCREASE IN VOL. 3.05 % AVERAGE DRY DENSITY AT FAILURE 91.4 pcf





RESEARCH COUNCIL OF ALBERTA  
DIVISION OF HIGHWAY RESEARCH  
SOIL STABILIZATION DATA SHEET

PROJECT \* SOIL - LIME - POZZOLAN  
SOIL DESCRIPTION Clay - Fahler - CH  
ACCELERATOR DESCRIPTION N11

TECHNICIAN gul. TEST NO. 5B

DATE MIXED June 19/61 DATE COMPACTED June 20/61 DATE END OF CURING July 18/61  
28 DAY FREEZE-THAW CONTROL ✓ DATE TESTED July 19/61  
% ADDITIVE 20 LIME-POZZOLAN RATIO 1:1 ACCELERATOR

Sample number	31	32	33	34	35	36
Compacted height ins.	2.048	2.057	2.050	2.065	2.056	2.046
Compacted weight gms.	195.92	196.28	196.24	196.30	196.14	195.75
Compacted wet density pcf.	116.0	116.0	116.2	115.2	115.8	115.2
Compacted volume c.i.	6.435	6.463	6.441	6.488	6.460	6.429
Total freeze-thaw cycles						
Diam. before compression	2.035	2.037	2.035	2.035	2.037	2.036
Area before compression	3.25	3.25	3.25	3.25	3.25	3.25
Length before compression	2.090	2.105	2.092	2.102	2.100	2.087
Volume before compression	6.799	6.860	6.805	6.838	6.844	6.795
Strain dial at failure	.438	.425	.438	.427	.428	.446
Strain dial init. reading	.363	.353	.356	.352	.354	.355
Strain INS/inch	.038	.036	.036	.038	.037	.045
Corrected area ins.	3.38	3.38	3.38	3.38	3.38	3.40
Load dial divisions	221.	198.	210.	213.	192.	214.
Pounds per division	4.00	4.00	4.00	4.00	4.00	4.00
Stress p.s.i.	261.	234.	248.	252.	227.	251.5
Increase in volume %	5.66	6.14	5.65	5.39	5.94	5.69
Wt. before compression gms.	208.40	209.22	208.50	209.22	208.91	208.12
Dry density at failure pcf.	89.7	89.1	87.7	88.8	88.6	89.8

MOISTURE CONTENT

No. container	C1	C2	C3	C4	C5	C6
Wt. sample wet + tare gms.	86.18	64.22	70.26	131.55	131.52	126.61
Wt. sample dry + tare gms.	71.52	54.54	59.40	109.88	110.14	106.42
Weight of water gms.	14.66	9.68	11.86	21.67	21.38	20.19
Tare container gms.	23.81	22.93	23.78	40.39	41.63	39.80
Weight of dry soil gms.	47.71	31.61	35.62	69.49	68.51	66.62
Moisture content %	30.7%	36.0%	33.3%	31.2%	31.2%	32.0%

AVERAGE STRESS 246 psi INITIAL MOISTURE CONTENT 23.6 %  
CORRECTED STRESS  psi AVERAGE COMPACTED DRY DENSITY 93.5 pcf  
AVERAGE INCREASE IN VOL. 5.75 % AVERAGE DRY DENSITY AT FAILURE 89.3 pcf



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1969  
1970

1971  
1972  
1973

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708	709	710	711	712	713	714
715	716	717	718	719	720	721
722	723	724	725	726	727	728
729	730	731	732	733	734	735
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743	744	745	746	747	748	749
750	751	752	753	754	755	756
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764	765	766	767	768	769	770
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778	779	780	781	782	783	784
785	786	787	788	789	790	791
792	793	794	795	796	797	798
799	800	801	802	803	804	805
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813	814	815	816	817	818	819
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862	863	864	865	866	867	868
869	870	871	872	873	874	875
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904	905	906	907	908	909	910
911	912	913	914	915	916	917
918	919	920	921	922	923	924
925	926	927	928	929	930	931
932	933	934	935	936	937	938
939	940	941	942	943	944	945
946	947	948	949	950	951	952
953	954	955	956	957	958	959
960	961	962	963	964	965	966
967	968	969	970	971	972	973
974	975	976	977	978	979	980
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988	989	990	991	992	993	994
995	996	997	998	999	1000	1001
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1107	1108	1109	1110	1111	1112	1113
1114	1115	1116	1117	1118	1119	1120
1121	1122	1123	1124	1125	1126	1127
1128	1129	1130	1131	1132	1133	1134
1135	1136	1137	1138	1139	1140	1141
1142	1143	1144	1145	1146	1147	1148
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1163	1164	1165	1166	1167	1168	1169
1170	1171	1172	1173	1174	1175	1176
1177	1178	1179	1180	1181	1182	1183
1184	1185	1186	1187	1188	1189	1190
1191	1192	1193	1194	1195	1196	1197
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1268	1269	1270	1271	1272	1273	1274
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1289	1290	1291	1292	1293	1294	1295
1296	1297	1298	1299	1300	1301	1302
1303	1304	1305	1306	1307	1308	1309
1310	1311	1312	1313	1314	1315	1316
1317	1318	1319	1320	1321	1322	1323
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1352	1353	1354	1355	1356	1357	1358
1359	1360	1361	1362	1363	1364	1365
1366	1367	1368	1369	1370	1371	1372
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1380	1381	1382	1383	1384	1385	1386
1387	1388	1389	1390	1391	1392	

RESEARCH COUNCIL OF ALBERTA  
DIVISION OF HIGHWAY RESEARCH  
SOIL STABILIZATION DATA SHEET

PROJECT \* SOIL - LIME - POZZOLAN  
SOIL DESCRIPTION Clay - Fahler - CH  
ACCELERATOR DESCRIPTION N11

TECHNICIAN gw TEST NO. 5B

DATE MIXED June 19/61 DATE COMPACTED June 20/61 DATE END OF CURING July 18/61  
28 DAY FREEZE-THAW ✓ CONTROL DATE TESTED July 19/61  
% ADDITIVE 20 LIME-POZZOLAN RATIO 5:5 ACCELERATOR ---

Sample number	41	42	43	44	45	46
Compacted height ins.	2.047	2.042	2.048	2.045	2.041	2.038
Compacted weight gms.	195.95	195.71	195.25	195.72	195.69	195.32
Compacted wet density pcf.	115.3	115.6	115.8	115.2	115.6	115.9
Compacted volume c.i.	6.43	6.42	6.43	6.43	6.42	6.40
Total freeze-thaw cycles	6.	6.	6.	6.	6.	6.
Diam. before compression	2.043	broke	2.054	2.046	2.039	2.045
Area before compression	3.28	in	3.31	3.29	3.27	3.29
Length before compression	2.140	half	2.162	2.140	2.148	2.142
Volume before compression	7.02	before	7.17	7.04	7.03	7.05
Strain dial at failure	.415	testing	.405	.411	.410	.418
Strain dial init. reading	.326		.301	.314	.316	.322
Strain INS/inch	.045		.052	.049	.047	.047
Corrected area ins.	---		---	---	---	---
Load dial divisions	110.		103.	114.	102.	123.
Pounds per division	4.44		4.17	4.61	4.13	4.97
Stress p.s.i.	135.		126.	140.	126.	151.
Increase in volume %						
Wt. before compression gms.	212.38	211.55	214.50	212.25	211.52	212.04
Dry density at failure pcf.						

MOISTURE CONTENT

Corner  
torn  
out

No. container	A1	A2	A3	A4	A5	A6
Wt. sample wet + tare gms.	141.41	163.25	104.41	106.78	131.98	86.48
Wt. sample dry + tare gms.	116.47	134.37	84.92	87.53	109.13	71.43
Weight of water gms.	24.94	28.88	19.49	19.25	22.85	15.05
Tare container gms.	41.21	43.04	29.50	26.49	40.51	26.34
Weight of dry soil gms.	75.26	91.33	55.42	61.04	68.67	45.09
Moisture content %	33.1	31.7	35.2	32.0	33.2	33.4

AVERAGE STRESS 136 psi INITIAL MOISTURE CONTENT 23.6 %  
CORRECTED STRESS --- psi AVERAGE COMPACTED DRY DENSITY 93.5 pcf  
AVERAGE INCREASE IN VOL. --- % AVERAGE DRY DENSITY AT FAILURE --- pcf  
 $R_f = \frac{136}{246} = 55.3$



RESEARCH COUNCIL OF ALBERTA  
DIVISION OF HIGHWAY RESEARCH  
SOIL STABILIZATION DATA SHEET

PROJECT \* SOIL - LIME - POZZOLAN  
SOIL DESCRIPTION CLAY - FAHLER - CH  
ACCELERATOR DESCRIPTION 0.5% Na<sub>2</sub>CO<sub>3</sub>  
Sodium Carbonate  
TECHNICIAN g.w. TEST NO. 7E

MIXTURE DESIGN..... <u>20</u> ..... <u>7</u> ..... <u>3</u> .....	For 1 sample	For 22 samples	MC %	Weight of water
Weight of dry mix gms.	139			
Weight of dry soil gms.	115.8	2540	25.5	674
Weight of dry soil + <u>5%</u> H <sub>2</sub> O gms.		2665		
Weight of additive gms.	23.2	510	31.5	160.5
Weight of lime gms.		356		
Weight of pozzolan gms.		153		
Weight of accelerator gms.		12.7		
Optimum moisture content <u>31.5%</u>				
Weight of water added gms.		834.5		
Weight of wet sample gms.	183			834.5

183 x 22 = 4020 4021.2

DATE MIXED. July 12/61 DATE COMPACTED July 13/61  
TYPE OF COMPACTION 5 BLOWS EACH END WITH 5 POUND WEIGHT  
AVERAGE WET DENSITY 107.8 pcf CURING TEMPERATURE MAX. \_\_\_\_\_ °C.  
AVERAGE DRY DENSITY 82.5 pcf CURING TEMPERATURE MIN. \_\_\_\_\_ °C.

	MOISTURE CONTENT	PLASTIC LIMIT
No. container		A14
Wt. sample wet + tare gms.	179.37	65.0140
Wt. sample dry + tare gms.	133.04	63.9635
Weight of water gms.	26.33	1.0505
Tare container gms.	47.97	61.0544
Weight of dry soil gms.	86.07	2.9091
Moisture content %	30.6%	36.0%

### RESULTS

Compressive strength after 28 days 161 psi.  
Compressive strength after 12 cycles freeze-thaw --- psi.  
Compressive strength of freeze-thaw control 231 psi.  
Remarks: Tops came off all freeze-thaw specimens, top one-quarter very rough and in loose condition.













**B29795**